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Evaluation of the Collapse Fragility Curve of the Moment Resisting Steel frames with Rigid and Semi-rigid Connections under Near-Field Earthquake Records

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Abstract
One of the major impacts of near-field earthquakes is a concentration of rupture on the limited number of stories and structural elements beyond the expected. Therefore, predicting the distribution of quantitative parameters of earthquake response at structural height can help to estimate the destructive potency of near-field earthquakes. In near-fault regions, directivity cause fling-step in the velocity time-history records, imposing more resistance and ductility requirements on the structure perpendicular to the fault line. The Endurance Time (ET) method is an innovative and straightforward method for dynamic loading and analysis of structures, apprehensible for the standard level of seismic engineering knowledge. The collapse performance and the accuracy of the ET in the seismic assessment of steel moment-resisting frames are discussed. Results of ET and IDA compared to observe the ET method's potential benefits and drawbacks in the seismic evaluation of this category of frames. To model the semi-rigid connection to reduce flexural stiffness, the width of the upper and lower beam flanges were reduced and the results were analyzed. According to the obtained results, the reduction in rigidity percentage decreases the median of collapse capacities and increases the dispersion of IDA curves and seismic vulnerability of the building. Also, it was observed that the ET method overestimates the median of collapse capacity and leads to unsafe design.

Keywords: Collapse Fragility Curve, Incremental Dynamic Analysis, Endurance-Time Analysis, Rigid and Semi-Rigid Connection.

1. Introduction

In the design of multi-story buildings, typical vertical loads are not problematic, but lateral loads due to wind or earthquake is of particular importance and require special attention. Near-field earthquakes are lateral loads that have distinctive features that distinguish them from far-field earthquakes. Near-field (NF) ground motions are specified by long-period velocity and displacement pulses [1] and high values of the ratio between the peak of vertical and horizontal ground accelerations [2]. The amplitude of this pulse depends on the directivity of rupture distribution to the site. Since the rupture propagation velocity is almost the same as the velocity of shear wave diffusion, if the fault rupture propagates to the considered place, the waves in a short-term period will reach to the place resulted in a pulse with high amplitude and short period that is called forward-effect directivity [3, 4]. Since then, numerous connections have been provided to retrofit and to redesign and improve steel flexural frames in the high-hazard level region. The occurrence of pulses at the beginning of the earthquake indicates the release of significant kinetic energy over a short period of time due to fault failure [3]. After the 1966 Parkfield earthquake in California and the 1971 Pacoima earthquake in San Fernando, the term near-fault was first suggested by Bolt in 1975 [5]; while the 1992 Landers earthquake, the 1994 Northridge earthquake, the 1995 Kobe

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earthquake and the 1999 Chi-Chi earthquake in Taiwan highlighted this term for civil engineers. These earthquakes, which occur near an active fault, have pulse mappings with a long pulse period and have one or more velocity peaks. In the near-fault region, the horizontal component perpendicular to the fault has the greatest effect on structural response and the effect of this component is dominant on the horizontal component parallel to the fault and on the vertical component earthquake record.

Progressive directivity effects cause horizontal ground vibrations perpendicular to the fault as a horizontal impact which is substantially larger than horizontal component parallel to the fault. As the angle between the fault and the site decreases and the failure level between the fault and site becomes larger, the effect of the progressive directivity effect becomes greater. Such pulses increase the nonlinear displacement demand in the structure so that near the fault they can impose large displacement to the structure near-fault [6]. Since the horizontal component perpendicular to the fault is dominant near-fault, the orientation of the structures located near fault is important. The ground acceleration variability often affects the stiffness of the structural element and connections type.

Akbas and Shen showed the semi-rigid connections would have a larger drift response than the rigid connections in the frames under weak to moderate stimulus on the foundation [7]. One of the methods used to estimate the performance of connections under the ground motion records is to obtain a collapse fragility curve of buildings with different rigidity percentages. The effect of semi-rigid connections on fragility curves of moment-resisting steel frames under Far-Field Earthquake Records was evaluated. results showed that decreasing the rigidity percentage in connections up to 50%, decreases the median of collapse capacities and increases the seismic vulnerability of the building [8].

The study of the global collapse was triggered by considering P-Δ effects on seismic response. Currently, the collapse fragility curve is the most important and accepted tool for evaluating the collapse of the structure. A set of IDA analyses can play a vital role in determining the estimation parameters and in turn determine the collapse fragility curve. Incremental Dynamic Analysis (IDA) and Endurance Time (ET) analysis are employed to take the inherent variability of earthquakes into account during the seismic response analysis of structures. Prevention of collapse and instability has always been one of the goals of seismic design. In the earthquake engineering, collapse and instability refer to the reduced capacity of structural systems to resist gravity loads during earthquake records. Assessing the safety against the collapse enforces to estimate the dynamic response of systems with the potential for stiffness and resistance losses [9-13]. This will lead to more sophisticated modelling and analysis techniques and will result in considerable uncertainties.

Since 2004, the Endurance Time (ET) method has been introduced as an alternative response method for the seismic analysis and structural design [14]. In this way, the computational demand is significantly reduced by subjecting the structure to an intensifying acceleration function and monitoring the objective performance indexes through time. Subsequently, structural performance can be evaluated based on the response of the system at each excitation level [15, 16]. Generating the appropriate artificial dynamic input is essential for the success of the ET method. Given this, an input function can be considered as appropriate if the estimated results in ET analysis are consistent with the performance of different structures under real earthquakes. The acceleration functions currently applied in the ET method have two specific properties: (I) these functions are intensifying as their amplitude increase with time, (II) these functions are optimized such that the response spectrum of any window from t=0 to t=t1 is proportional to a template response spectrum with a scale factor that linearly varies with time [15]. In the endurance time method, the structure is subjected to an incremental accelerometer and the maximum value of response parameters is plotted versus time. Depending on the need, these response parameters may include one or a set of performance criteria used in evaluation and design. It is common for steel structures to idealize moment connections as rigid. The rigid connection is a connection between beam and column which prevents the beam rotation. Also, in semi-rigid connection transfer moment but allow the joint to move. In this study, the application of the ET procedure in the collapse analysis of steel frames is investigated.

For this purpose, three moment-resisting are considered. Results of ET and IDA nonlinear analyses and collapse performance of these frames are discussed and compared to observe the ET method’s potential benefits and drawbacks in the seismic evaluation of this category of frames. Considering a number 5-, 10-, and 15-story, 2-D, steel moment-resisting buildings, four percent of rigidity and two methods is employed to estimate their collapse fragility curve. On the other hand, the effect of the rigidity percentage and ET method on the collapse fragility curves were taken into account by considering the near-field ground motion records.

2. Research Methodology

2.1. Collapse Fragility Curve

The collapse in this context is defined as the loss of lateral load-resisting capability of a building's structural system caused by ground shaking. Estimation of collapse performance requires the relation between a ground motion intensity measure (IM) and the probability of collapse, denoted as collapse fragility curve. To evaluate the collapse performance of structures, the collapse fragility curve is the most important and acceptable tool for this purpose. The collapse fragility curve expresses the probability of structural collapse at different levels of ground-motion intensity measure (IM). The collapse fragility curve can be defined by probability distribution function as [1]:

$$P[C|IM] = FC_{IMc}(X) = P[IM \geq IM_c \,|\, IM = x] = P[IM_C \leq x] = \Phi[\frac{\mathrm{Ln}\, IM - Ln\eta_C}{\beta_{RC}}] \qquad (1)$$

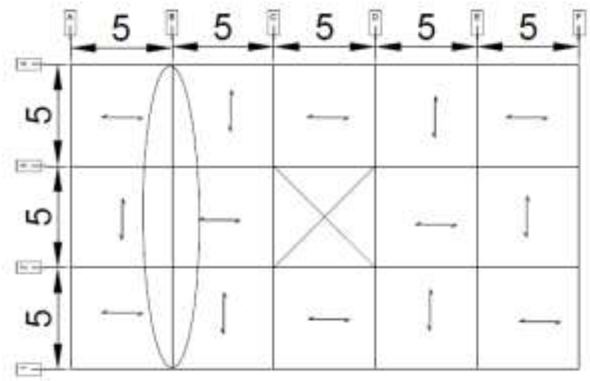
Where $FC_{IMc}(X)$ is a point on the fragility curve (FC) at IM=x in the collapse state. Given that seismic demand (x) is probabilistically independent of system capacity (S_{ac}), therefore the fragility function can be presented as the probability that S_{ac} is less than or equal to x.

2.2. The Effects of Uncertainty on Probability of Collapse

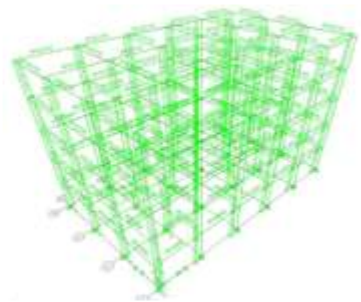
The uncertainty in IMc estimation in the IM-based method and EDPd and EDPc estimation in the EDP-based method is due to the random nature of ground motion. Two approximations can be used to combine the effects of two types of uncertainties in estimating the probability of collapse at a given IM: "confidence level" and "mean method". In the confidence level method, the goal is to find the collapse fragility curve with a certain degree of confidence, such as Y, which is the probability that the mean value (denoted as $\eta^{Y}_{\acute{C}_{\iota}}$) of the collapse capacity is greater than ηˆC. The collapse fragility curve of the confidence level Y is a normal logarithmic distribution whose mean is $\eta^{Y}_{\acute{C}_{\iota}}$ and its dispersion is β_{RC}. In the mean method, it is assumed that, to combine the effects of epistemic and aleatory uncertainties in estimating the probability of collapse, the effects of two sources of variability can be centralized on the dispersion of collapse fragility curve (denoted by β_{TC}). Therefore, in the mean method, the collapse fragility curve is defined as the logarithmic normal distribution with the mean $\hat{\eta}_c$ and the dispersion $\beta_{TC} = \sqrt{{\beta_{RC}}^2 + {\beta_{UC}}^2}$. For further explanation of this approach, the reader is referred to as Zareian and Krawinkler's (2007) [13]. This method is also used in the evaluation guidelines in FEMAP695 [17].

2.3. The Considered Structural Models

Three steel frames with rigid and semi-rigid connections subjected to the 7 Near-Field ground motions records with the fundamental periods of 0.884, 1.272 and 1.9156 seconds were considered and designed according to the ASCE 7-10 code requirements [18]. The selection of the building is based on the different period range which contains the short-to-high rise buildings. These structural models are assumed to be of administrative buildings type with the same plan dimensions, located in a high-hazard seismic level of Tehran with site class D and designed according to the ASCE 7-10 using Etabs ver.9.7.0 software [19]. The seismic parameter A was considered 0.35, respectively; the importance factor (I) of 1, the response modification factor (R) of 5 (5- and 10-story) and 7.5 (15-story) were considered. The structural system of the 6- and 12-story frames (S-5 and S-10) is intermediate moment resisting frames, while the 18-story frame (S-15) is considered as the special moment-resisting frame. The beam and column sections were selected I-shape and box-shape, respectively. The steel type is ST-37, Fy = 2400 kg/cm2 and Fu = 3700 kg/cm2. The studied buildings have a 25 m×15 m rectangle plan with a story height of 4.0 meters and spans of 5 meters. In designing the building models the story drift ratios were limited to values specified by the considered code. Figure 1 shows the typical plan of buildings and the selected frame was considered in this study. Also, section properties of structural elements present in tables 1-3.



(a)



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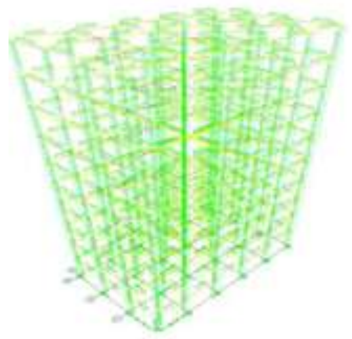

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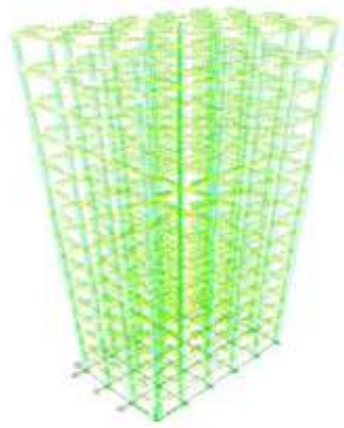


(d)

Fig. 1. Structural plan, elevation and selected frame
(a) 6-story building, (b) 12-story building, (c) 18-story building, (d) typical plan

Table 1
Section properties of rigid 5-story frame structures

NO	*STORY*	*COLUMN*	*BEAM*
1	S-1	Box 400*400*12	PG360T10F250T20
2	S-2	Box 400*400*12	PG360T10F250T20
3	S-3	Box 350*350*12	PG360T10F250T20
4	S-4	Box 350*350*12	PG330T8F250T15
5	S-5	Box 300*300*10	PG330T8F250T15

Table 2
Section properties of rigid 10-story frame structures

NO	*STORY*	*COLUMN*	*BEAM*
1	S-1	Box 450*450*20	PG400T12F250T20
2	S-2	Box 450*450*20	PG400T12F250T20
3	S-3	Box 450*450*20	PG400T12F250T20
4	S-4	Box 400*400*15	PG400T12F250T20
5	S-5	Box 400*400*15	PG350T10F250T20
6	S-6	Box 400*400*15	PG350T10F250T20
7	S-7	Box 400*400*15	PG350T10F250T20
8	S-8	Box 400*400*15	PG330T10F200T15
9	S-9	Box 350*350*12	PG330T8F200T12
10	S-10	Box 350*350*12	PG330T8F200T12

Table 3
Section properties of rigid 15-story frame structures

NO	*STORY*	*COLUMN*	*BEAM*
1	S-1	Box 500*500*20	PG450T10F250T20
2	S-2	Box 500*500*20	PG450T10F250T20
3	S-3	Box 500*500*20	PG450T10F250T20
4	S-4	Box 500*500*20	PG450T10F250T20
5	S-5	Box 500*500*20	PG450T10F250T20
6	S-6	Box 500*500*20	PG450T10F250T20
7	S-7	Box 500*500*20	PG400T10F250T20
8	S-8	Box 450*450*20	PG400T10F250T20
9	S-9	Box 450*450*20	PG400T10F250T20
10	S-10	Box 450*450*20	PG400T10F250T20
11	S-11	Box 400*400*15	PG400T10F250T20
12	S-12	Box 400*400*15	PG350T10F250T20
13	S-13	Box 400*400*15	PG350T10F250T20
14	S-14	Box 400*400*15	PG350T10F250T20
15	S-15	Box 400*400*15	PG350T10F250T20

2.4. Near-Field Selected Earthquake Records

For nonlinear Time History Analysis (THA), 7 pairs of near-field earthquake records, most of them used in the ATC-58 and FEMA 440 [20], were extracted from the PEER [21]. Table 4 lists the characteristics of 7 earthquake record pairs recorded on very dense soil (shear wave velocity 375 m/s to 750 m/s) that have a magnitude of 5.5 to 7.5 and a fault distance in the range of 7 to 20 km. The selected records were normalized according to the ASCE 7-10 code [18] before being used in the extensive nonlinear dynamic time-history analyses.

Table 4
Specification of earthquake records for the numerical analyses [21]

No	Earthquake Name	Station number	Year	Magnitude (Ms)	R-JB(km)	R-rup(km)	Vs(m/s)	EFF-Time (sec)
1	Big bear	901	1992	6.46	7.31	8.3	430.36	0-60
2	Kobe japan	1111	1995	6.9	7.08	7.08	609	0-40
3	Loma prieta	739	1989	6.93	19.9	20.26	488.77	0-78
4	N.Palm springs	518	1986	6.06	12.79	14.24	388.63	0-40
5	Northriage	957	1994	6.69	15.87	16.88	581.93	0-30
6	Nrkfield	4143	2004	6	9.14	9.61	440.59	0-120
7	Sanfernando	72	1971	6.61	9.451	25.07	600.6	0-35

2.5. Numerical Modelling

To evaluate the performance of a structure, an analytical method should be used to determine the structural responses at all functional limits. For this purpose, two methods are performed: Incremental Dynamic Analysis (IDA) and Endurance Time Analysis (ET). Dynamic time history analysis is commonly used to estimate nonlinear structural behaviour during earthquakes. This study uses direct integration of motion equation and the optimized time interval adjustment algorithm. This algorithm improves numerical convergence conditions and speeds up the dynamic nonlinear analysis. Given the complexities of natural earthquake selection and the random nature of earthquake acceleration records, some scientists tended to produce synthetic earthquakes that can be produced for predetermined purposes. In this regard, specific control objectives such as equal level of energy applied to structures from natural and synthetic earthquakes as well as the duration of strong ground acceleration and adaptation of artificial response spectrum to natural mean response spectrum of earthquakes are discussed. For this purpose, various synthetic records have been made by many scientists in the field of structural engineering and earthquake. One of the best sets of synthetic records made in this regard is endurance-time synthetic records, which are directly used in the endurance-time analysis.

The elastic spectral acceleration in the fundamental period of a structure for 5% of critical damping is considered as intensity measure (IM). The intensity measure was applied to the structures from zero to the intensity leading to the collapse of the structure. Seismostruct 2016 software can consider nonlinear behaviours caused by changes in structural geometry and change in material properties. Time history analysis to determine structural response under time-dependent loads is performed by stepwise numerical integration of motion equations. In this study, structural analysis is performed using Seismostruct 2016 software.

3. Analysis and Results

After selecting and scaling the earthquake records and modelling the studied frames, incremental dynamic analysis (IDA) was performed under the horizontal component of the selected records using Seismostruct2016 software. To obtain IDA curves, the IM-based method was used. Figures 2 to 4 present the IDA curves of the 5-, 10- and 15- story frames under the horizontal components of the selected earthquake records for the cases rigid connections and semi-rigid connections corresponding to 50% to 70% rigidity.

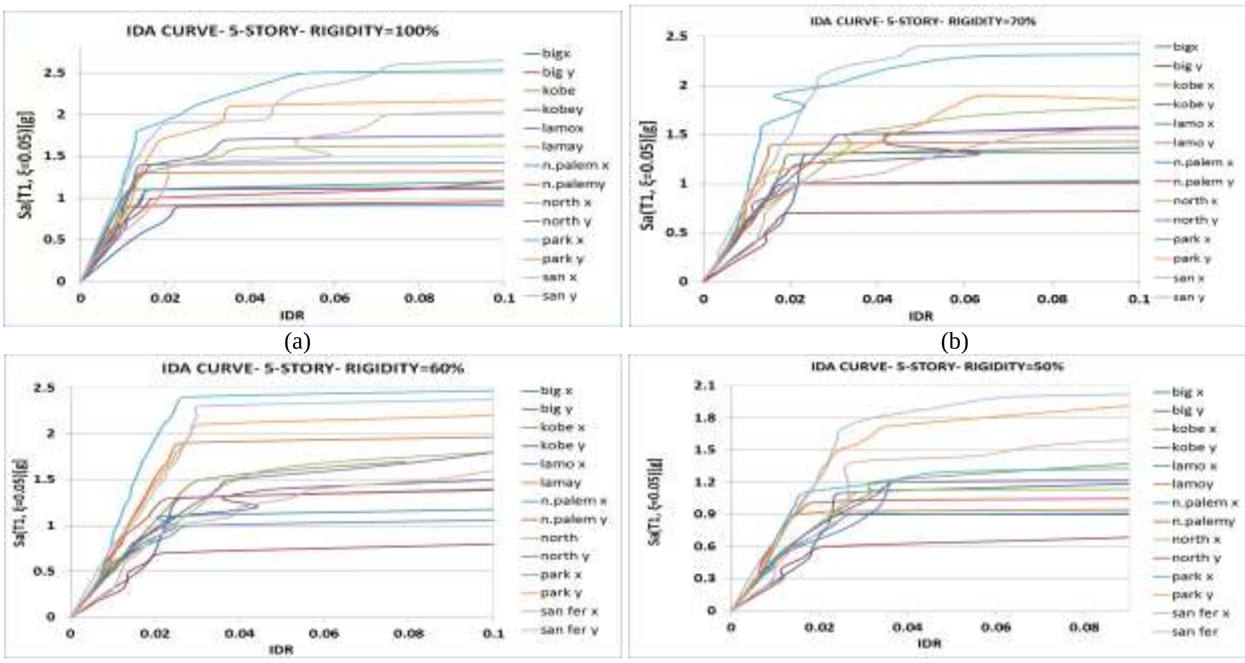


Fig 2. IDA curves of the 5-story frame in four states: (a) Rigid connections, (b) Semi-rigid connections (70% rigidity), (c) Semi-rigid connections (60% rigidity), (d) Semi -rigid connections (50% rigidity)

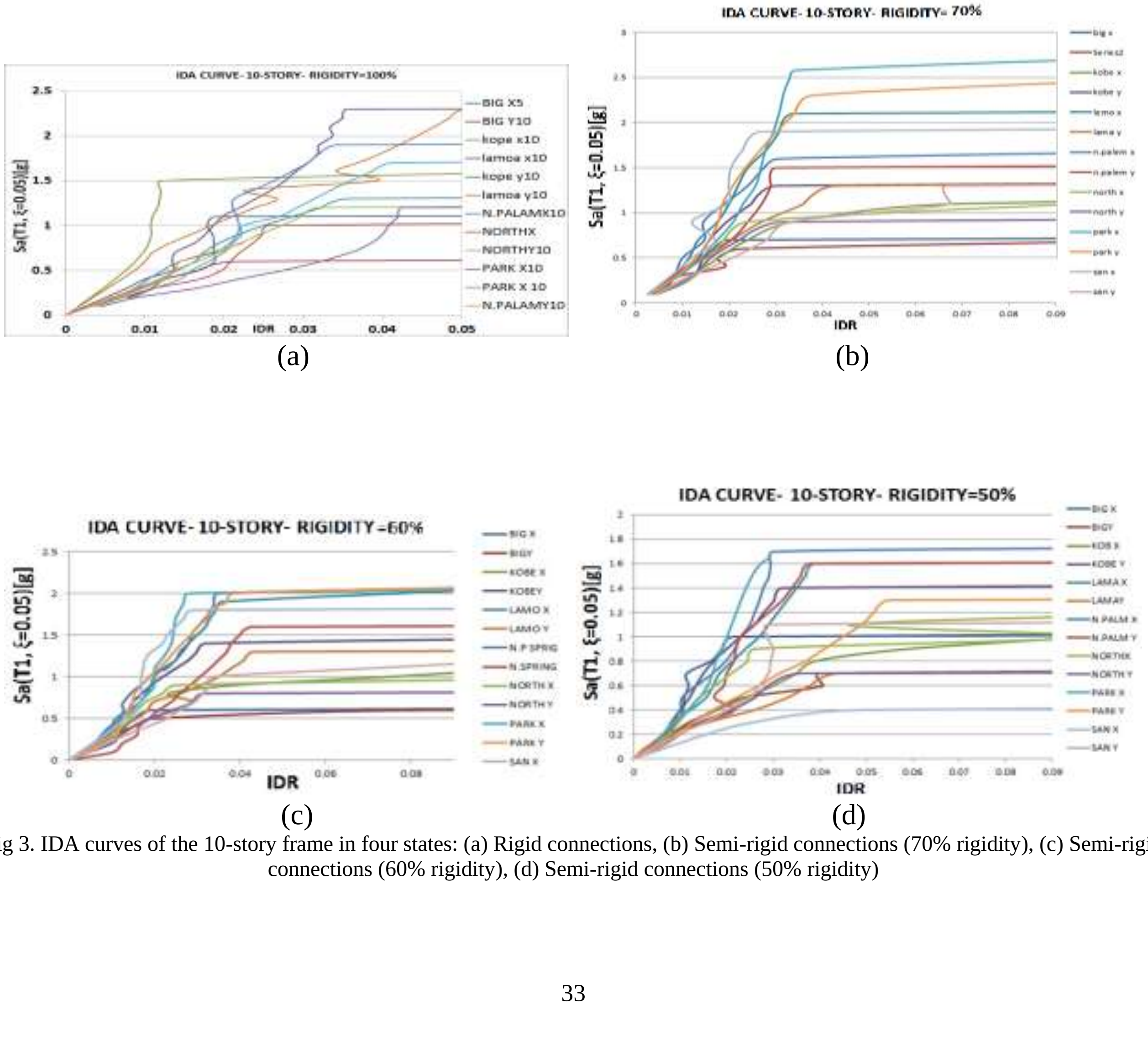


Fig 3. IDA curves of the 10-story frame in four states: (a) Rigid connections, (b) Semi-rigid connections (70% rigidity), (c) Semi-rigid connections (60% rigidity), (d) Semi-rigid connections (50% rigidity)

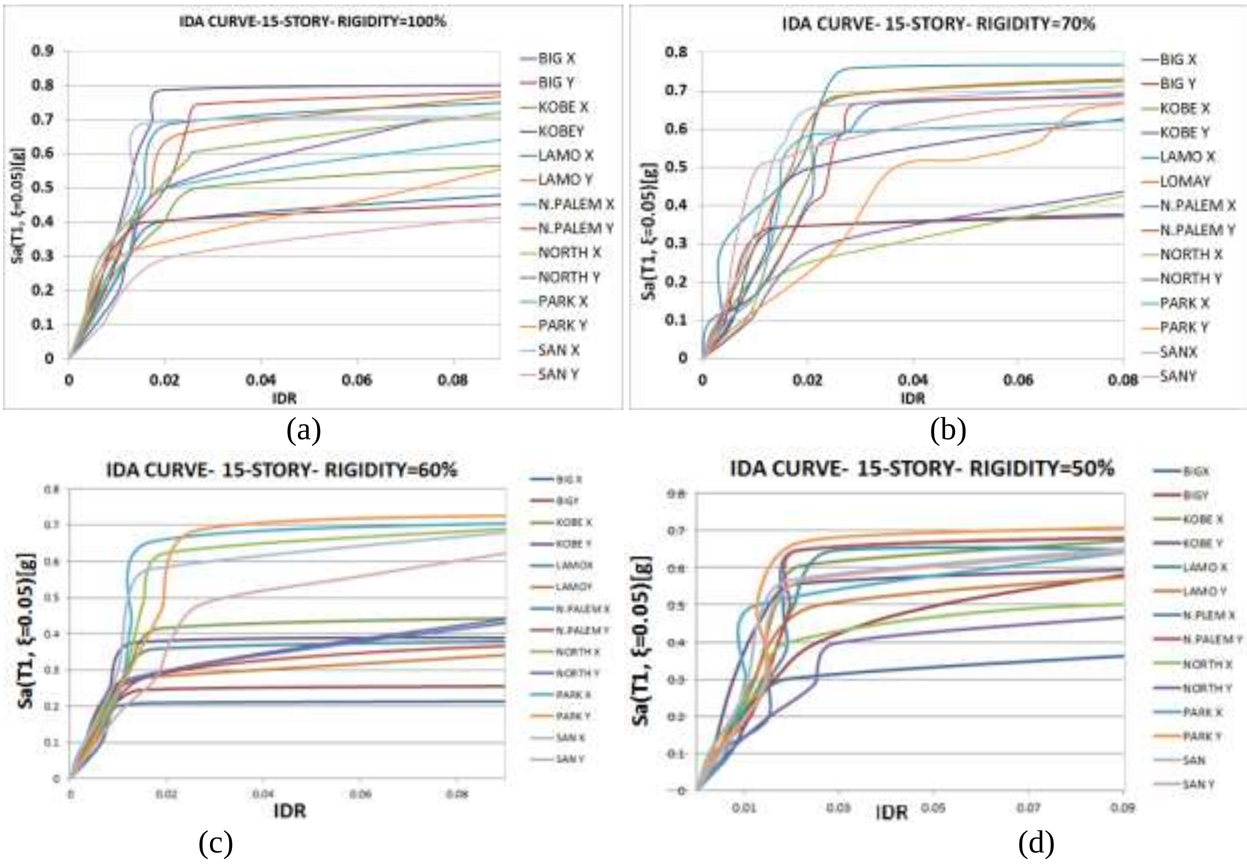


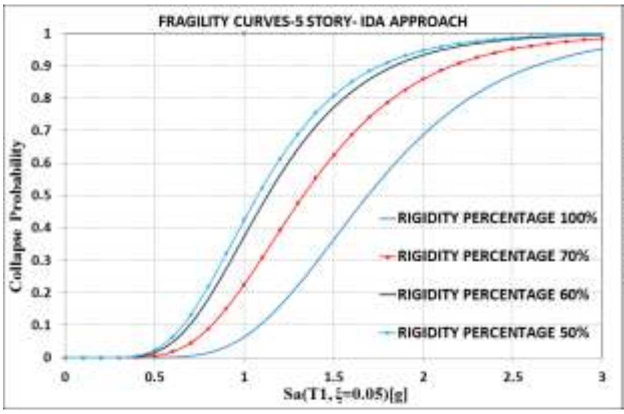
Fig 4. IDA curves of the 15-story frame in four states: (a) Rigid connections, (b) Semi-rigid connections (70% rigidity), (c) Semi -rigid connections (60% rigidity), (d) Semi -rigid connections (50% rigidity)

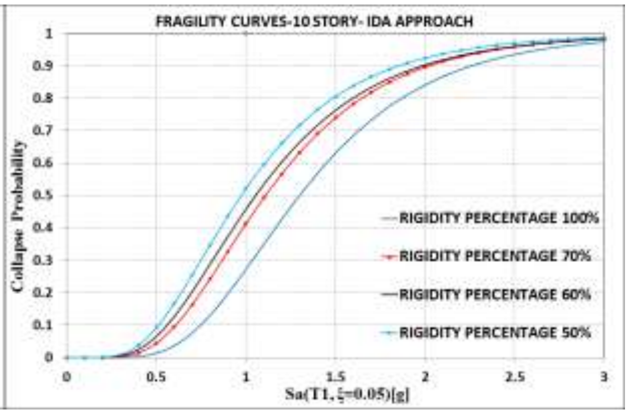
3.1. Determination of the Collapse Fragility Curves for the Studied Frames by IDA Method

Zareian [13] showed that the IM-based approach can be estimated the collapse fragility curve with better accuracy versus the EDP-base approach. By using an eight-story moment-resisting frame case study, he showed that the EDP-based method can be led to an overestimation in the probability of collapse under a certain level of the ground motion intensity and mean annual frequency of the collapse. Therefore, the collapse fragility curve of the studied buildings predicted by the IM-based approach. Ibarra and krawinkler showed that Sac points follow a log-normal distribution i.e. $\mathrm{Ln}(\mathrm{S_{ac}}) \rightarrow \mathrm{N}(\eta_C, \beta_{RC})$ where η_C and β_{RC} are median collapse capacity and dispersion of collapse capacity values due to different earthquake records which are numerically equal to the standard deviation of collapse capacity values [22]. For a given hazard level, such as PR, corresponding spectral acceleration can be obtained using seismic hazard curves and collapse probability can be calculated from Equation (2), where η_C and β_{RC} are median and standard deviation of the log-normal cumulative distribution function, respectively:

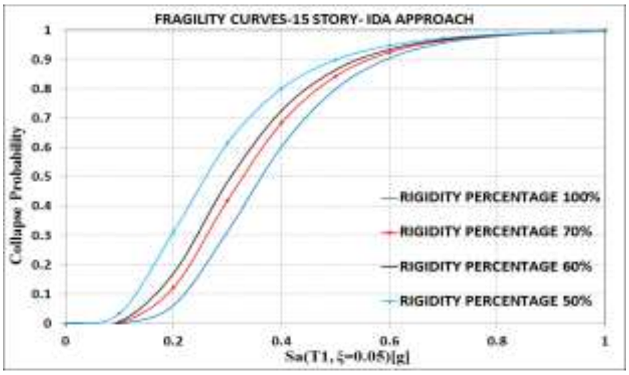
$$P(C|S_a^{PR}) = \Phi(\frac{Ln(S_a^{PR}) - Ln(\eta_C)}{\beta_{RC}}) \qquad (2)$$

Figures 5a, 5b and 5c present the fragility curves of the studied buildings using the IDA method in different rigidity percentages of connections.





(a) (b)



(c)

Fig 1: Fragility curves of the studied frames obtained using the IDA method in different rigidity percentage of connections: (a) 5-story frame, (b) 10-story frame and (c) 15-story frame

Since the above curves are in the form of log-normal cumulative distribution function with median (η_C) and dispersion (β_{RC}) parameters, their values are summarized in Table 5.

Table 5
Fragility Curves Parameters of the Studied Frames Obtained IDA Method

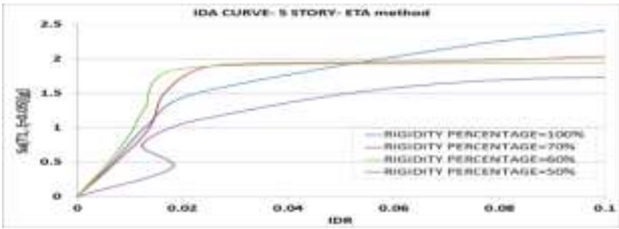
Rigidity percentage of connections	CDF Parameters	IM=Sa(T_1,ζ=0.05)		
		5-story	10-story	15-story
100%	median(η)	1.69	1.299	0.362
	STDEV(β_{RC})	0.344	0.432	0.386
70%	median(η)	1.33	1.109	0.327
	STDEV(β_{RC})	0.379	0.469	0.423
60%	median(η)	1.13	1.054	0.306
	STDEV(β_{RC})	0.38	0.494	0.447
50%	median(η)	1.075	0.974	0.258
	STDEV(β_{RC})	0.383	0.502	0.52
Error estimation of rigid-connections		21.3% to 36.4%	14.6% to 25%	9.6 % to 28.7%

Table 5 presents that decreasing the rigidity percentage of the connections in all stories from 100% to 50% decreases the median of the collapse fragility curve by 9.6%-36.39% and increases the dispersion of the collapse fragility curve. By comparison of the fragility curves of the studied frames, it can be concluded that the standard deviation of the fragility curve increases and the median collapse of the fragility curve decreases as height.

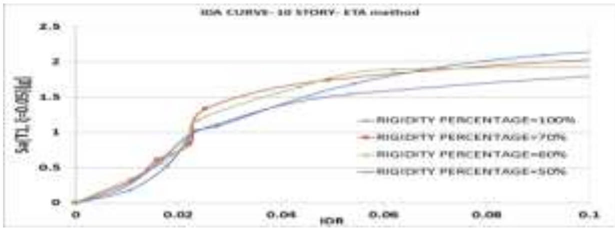
3.2. Plotting IDA Curve by ET Method

The Endurance Time (ET) method is an innovative and straightforward method for dynamic loading and analysis of structures, apprehensible for the standard level of seismic engineering knowledge. Three sets of second-generation Endurance Time Acceleration Functions (ETAFs) were used as input in the ET method. To plot the IDA curve in the ET method, each of the IDA curves corresponding to three Endurance Time Acceleration Functions (x1, x2, and x3) was calculated. T hen, the average of spectral accelerations and average of max internal drift ratio (IDR) at equal times were determined. Then, at different ts with equal or unequal intervals having average points of Sa and corresponding IDR, the average curve of three sets of ETAFs, called the IDA curve in the ET method, is obtained.

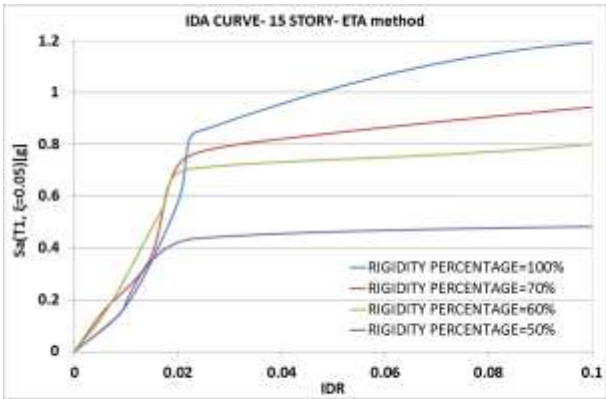
Figures 6(a) to 6(c) show the IDA curves of the 5-, 10- and 15- story frames in the ET method for the cases rigid connections and semi-rigid connections corresponding to 50% to 70% rigidity.



(a)



(b)



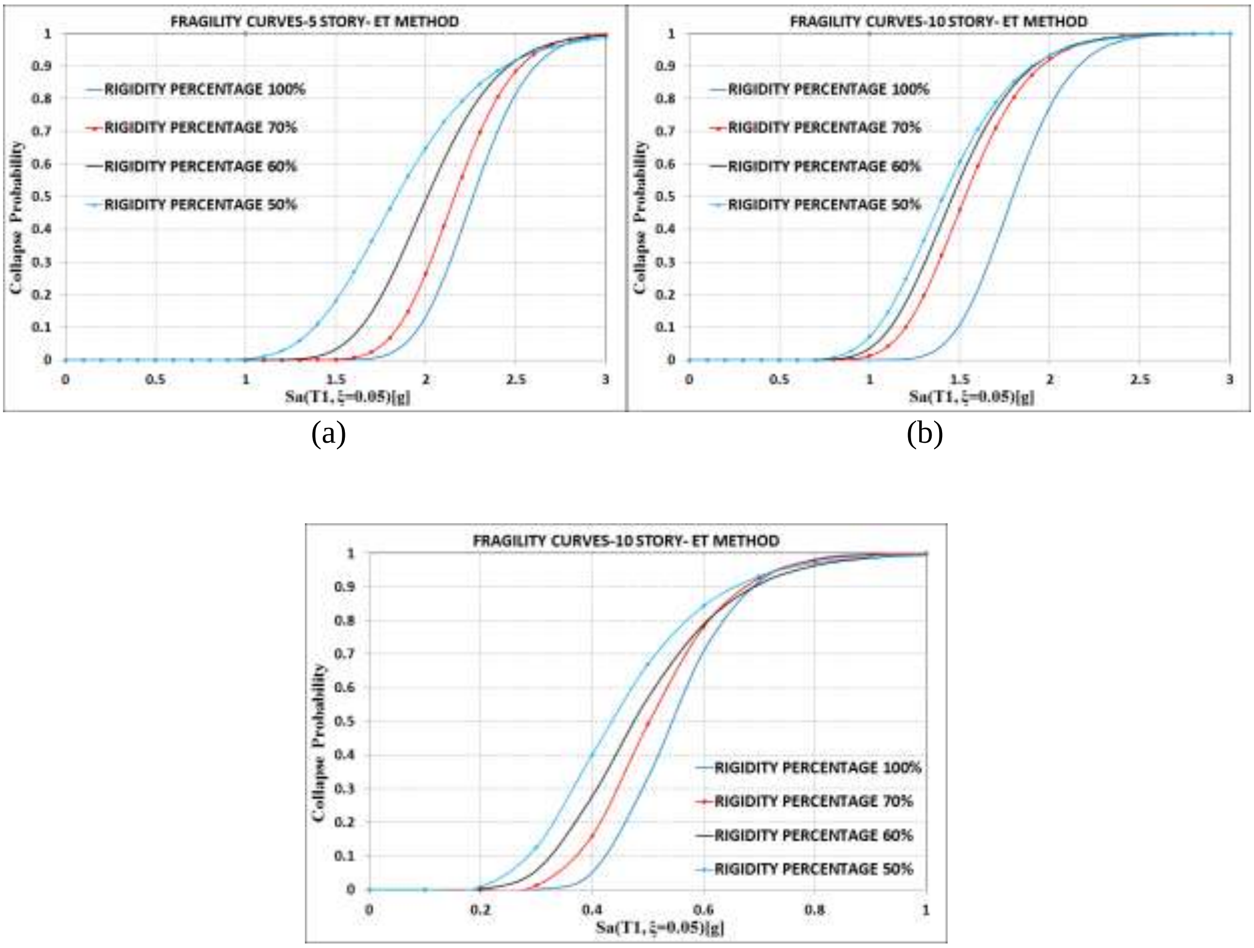
(c)

Fig 6. IDA curve in ETA method for studied frames: (a) 5-story frame, (b) 10-story frame and (c) 15-story frame in different rigidity percentage of connections

Inspecting figures 6 shows that decreasing the rigidity percentage of connections in the studied frames, reduced the collapse capacity. Also, results show that the ET method compared to the IDA method overestimates the collapse capacities of studied frames.

3.3. Determination of the Collapse Fragility Curves for the Studied Frames by ET Method

In this section, the collapse fragility curves of the studied building were obtained using the ET method. Figures 7 present the collapse fragility curves of the studied frames based on the ET approach under the second generation Endurance Time Acceleration Functions (x_1, x_2 and x_3). Since the fragility curves are in the form of the log-normal cumulative distribution function with median (η_C) and standard deviation (β_{RC}) parameters, their values are summarized in Table 6.



(a) (b)

(c)

Fig 7: Fragility curves of the studied frames obtained using the ET method in different rigidity percentage: (a) 5-story frame, (b) 10-story frame and (c) 15-story frame

Table 6 presents that decreasing the rigidity percentage of the connections in all stories from 100% to 50% decreases the median of the collapse fragility curve by 4.5%-21.4% and increases the dispersion of the collapse fragility curve. By Inspecting figures 6 and 7 shows that the IDA method overestimated the values for the median of the collapse fragility curve compared with the ET method and estimated the lower values for the dispersion, so it can be concluded that IDA method can lead to more sufficient results. Moreover, it can be concluded that second-generation Endurance Time Acceleration Functions are suitable in the nonlinear analysis under near-field earthquake records.

comparison of the fragility curves of the studied frames, it can be concluded that the standard deviation of the fragility curve increases and the median collapse of the fragility curve decreases.

Table 6
Fragility curves parameters of the studied frames obtained from ET method

Rigidity percentage of connections	CDF Parameters	IM=Sa(T_1,ζ=0.05)		
		5-story	10-story	15-story
100%	median(η)	2.262	1.79256	0.5421
	STDEV(β_{RC})	0.1109	0.1442	0.1858
70%	median(η)	2.159	1.5292	0.5025
	STDEV(β_{RC})	0.1222	0.1898	0.2291
60%	median(η)	2.007	1.4597	0.4752
	STDEV(β_{RC})	0.1582	0.21	0.2914
50%	median(η)	1.8356	1.40825	0.4338
	STDEV(β_{RC})	0.2219	0.2342	0.3223
Error estimation of rigid-connections		4.5% to 18.8%	14.6% to 21.4%	7.3 % to 19.9%

4. Conclusion

Assuming non-linear behavior for steel materials, this study modeled three 5-, 10- and 10- story steel moment-resisting frames. Incremental dynamic analysis (IDA) and endurance-time (ET) method were conducted to take the uncertainties of percentage rigidity and earthquake records into account. The building's performance was studied for rigid and semi-rigid connections using seismic demand probabilistic analysis. Besides, the effect of the different rigidity percentage of connections in the collapse fragility curve was evaluated. It was found that ET analysis can estimate THA results in an equivalent target time and also the general trend of IDA curves with acceptable accuracy, while ET requires considerably less computational effort in comparison with the IDA method. By examining the IDA curves related to the ET method, it can be concluded that as rigidity percentage was reduced, the median of collapsing capacity of the fragility curve for semi-rigid connections is reduced compared to the rigid connections.

Moreover, comparing ET and IDA methods shows that in the 5-,10- and 15-story buildings, ET analysis overestimates the median of the collapse capacities in the range of [25%-43%], [27%-30%] and [33%-40%], respectively This means that the ET method provides more conservative results than the IDA method.

It should be noted that the difference between the results of these two mentioned methods is less than 50% for studied buildings. Despite the overestimation of the collapse capacity by the ET method, the analysis time of the ET method is much shorter than the nonlinear time history method.

Therefore, the ET method is a good idea for performance-based analysis. Finally, IDA curves of studied buildings under the far-field earthquake records tend to have a slight slope ratio to near-field, thus structural deformation is low in near-field ground motion records compared to far-field. Note that the results of this study are without considering the epistemic uncertainty.

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