

EVALUATION AND COMPARISON OF THE RISK AND PROBABILITY OF COLLAPSES IN CONCRETE HIGH-RISE BUILDINGS WITH SHEAR WALLS IN THE NEAR AND FAR FIELDS

Sanaz ABDI¹, Mohsen TEHRANIZADEH² & Hamed GHASEMI³

Abstract: Interest in the possibility of a dependable method for earthquake prediction has increased among the scientific community, and the program focuses on both the prediction of shaking and the mitigation of risk. However, it is necessary to consider the earthquake with a probabilistic approach and take into account all uncertainties when calculating the risk in the design of existing buildings. This study aims to probabilistically assess and compare the seismic collapse risk of concrete high-rise buildings with shear walls in the near and far-field regions. To achieve this, a 20-story RC moment frame with the shear wall was initially designed according to the Code for Design of Concrete Structures (ACI-318) and the Code for Seismic Design of Structures (ASCE07-10). To investigate the effect of this building and the necessity of calculating fragility curves, nonlinear modeling was constructed using OpenSees program, considering the Fiber Beam-Column model for shear walls. The seismic collapse risk of the building was evaluated through incremental dynamic analysis (IDA) using a far and near-field database based on the FEMA P695 ground motion set, each of which includes 14 record pairs. The fragility curves and, consequently, the probability of uniform collapse risk in 50 years were evaluated. A detailed assessment of seismic collapse risk shows that the predicted probability of collapse in 50 years at the near-fault site is significantly higher than the possibility in the far-field region.

Introduction

The collapse of structures severely harms public safety and people's lives and property. Since the primary goal of seismic provisions in building codes is to protect life safety, collapse risk assessment becomes an essential step in preventive management. It helps identify areas at higher risk of collapse and select appropriate prevention approaches to reduce losses. Once the structural capacity is determined, considering the maximum acceleration, the concept of risk comes into play. Different definitions of risk exist for earthquake, including the risk of economic damage caused by earthquakes entering a building. In this case, risk refers to the probability of structural failure in a specified area within a particular time frame. The desired risk, as per US regulations, is one percent probability of building collapse in 50 years, and the acceleration corresponding to this risk is considered the maximum possible acceleration.

Taking a general overview of the studies conducted on assessing the risk of building collapse, they can be classified into four categories:

- Studies focusing on a single system (old or concrete buildings) throughout the region, aiming to gather data on collapse likelihood.
- Studies that examine collapse of a specific system and provided related fragility curves.
- Studies investigating composite concrete ductile frames and non-ductile factors that contribute to collapse, utilizing dynamic analysis.
- Studies that analyse fragility curves through nonlinear and dynamic analysis following an earthquake event.

This study quantifies and compares the seismic collapse risk of concrete high-rise buildings with shear walls in the near and far-field regions through dynamic analysis of nonlinear structural simulation models. The collapse of each structure is predicted using incremental dynamic analysis, where each ground motion is scaled to increasing intensity until dynamic instability

¹ Masters graduate of the Amir Kabir University of Technology, Department of Civil and Environmental Engineering, Tehran, Iran. (s.abdi.civil@gmail.com)

² Professor of the Amir Kabir University of Technology, Department of Civil and Environmental Engineering, Tehran, Iran.

³ Masters graduate of Maraghe University, Department of Civil Engineering, Tabriz, Iran.

occurs. To gain a better understanding of the seismic collapse risk of the structure, fragility functions representing structural collapse resistance are combined with recently developed probabilistic seismic hazard assessments to quantify the risk of earthquake-induced collapse in the near and far-fields regions. Considering all uncertainties, the uniform probability of collapse is calculated based on a one percent probability of failure for all the buildings and areas within 50 years. This study thus aims to probabilistically assess and compare the seismic collapse risk of concrete high-rise buildings with shear walls in the near-field and far-field regions. To achieve this, a 20-story RC moment frame with the shear wall structure was initially designed according to the Code for Design of Concrete Structures (ACI-318) and the Code for Seismic Design of Structures (ASCE07-10). To investigate the effect of this building and the necessity of calculating fragility curves, a nonlinear modeling was constructed using OpenSees program, taking into account the equivalent column model for shear walls. The far and near-fields database were based on the FEMA P695 ground motion set, each consisting of 14 record pairs.

Assessment of seismic risk in the region

Risk assessment approach

The approach for assessing the seismic risk involves evaluating the risk based on different definitions. In this case, the risk is defined as a one percent probability of a building collapse in within a 50-year period. To assess the risk of earthquake-induced collapse, the results of structural analysis are combined with probabilistic seismic hazard analysis (PSHA). PSHA helps determine the frequency at which a specific ground motion intensity measure (IM) could occur at a site of interest. By integrating these results, the possibility of structural collapse within a year is determined. Finally, using probabilistic relationships, this probability is converted into the probability of structural collapse within a 50-year period. The probability of collapse in 50 years is calculated assuming a Poisson distribution of earthquake occurrences.

$$P(\text{Collapse in } Y \text{ year}) = 1 - (1 - P[\text{Collapse}])^Y \quad (2)$$

A review of seismic hazard analysis

Earthquakes are natural hazards characterized by their random nature. Therefore, accurately determining the effects of future earthquakes is challenging, and probabilistic methods are employed to predict these effects to some extent. Seismic hazard analysis involves research and techniques aimed at estimating the impact of future earthquakes on a particular site. This impact is primarily determined by ground motion parameters. This analysis ultimately results in the development of an earthquake hazard curve in the site, which provides a spectrum of site-specific designs. These designs form the basis for the seismic design of structures against earthquakes.

In this study, the seismic design follows the ASCE-07 standard, which requires a site-specific seismic hazard curve for a given area in the United States. The USGS site was determined by specifying the site location and selecting site class C, represented very stiff soil sites. A specific point from the western United States was chosen for this soil type based on its shear wave velocity. Figure 1 illustrates the seismic hazard analysis curve obtained by considering the soil type in the region.

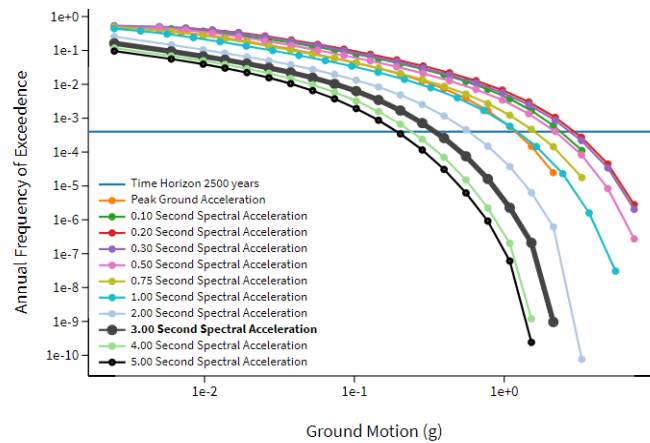


Figure 1. seismic hazard analysis curve.

Ground motion databases

Far-field record set

The far-fields database based on the FEMA P695 ground motion set. It consists twenty-two records (44 individual components) selected from the PEER NGA database. However, considering the site characteristics and shear wave velocity, it was found that fourteen of these ground motion records correspond to the soil type of the study area. Table 1 provides summary of the magnitude, year, and event name, as well as the station name and owner for each record.

NO.	Earthquake			Recording Station	
	M	Year	Name	Name	Owner
1	6.7	1994	Northridge	Beverly Hills-Mulhol	USC
2	6.7	1994	Northridge	Cayon Country- WLC	USC
3	7.1	1999	Duzce, Turkey	Bolu	ERD
4	7.1	1999	Hector Mine	Hector	SCSN
5	6.5	1979	Imperial Valley	Delta	UNAMUD
6	6.5	1979	Imperial Valley	El Centro Array #11	USGS
7	6.9	1995	Kobe, Japan	Nishi-Akashi	CUE
8	6.9	1995	Kobe, Japan	Shin-Osaka	CUE
9	7.5	1999	Kocaeli, Turkey	Duzce	ERD
10	7.5	1999	Kocaeli, Turkey	Arecelik	KOERI
11	7	1992	Cape Mendocino	Rio Dell Overpass	CDMG
12	7.6	1999	Chi Chi, Taiwan	Tcu045	CWB
13	6.6	1971	San Fernando	La-Hollywood staor	CDMG
14	6.5	1976	Friuli, Italy	Tomezzo	...

Table 1. Earthquake Event and Recording Station Data for the Far-Field Record Set.

Near-field record set

The Near-Field database is based on the FEMA P695 ground motion set. It consists of twenty-eight records (56 individual components) selected from the PEER NGA database. The near-field record set is divided into two subsets: The Pulse subset, which includes fourteen records with pulses, and the No-Pulse subset, which includes fourteen records without pulses. The classification was determined through wavelet analysis of the data.

However, considering the site characteristics and shear wave velocity, it was found that fourteen of these ground motion records (including both pulse and no-pulse subsets) correspond to the

soil type of the study area. Table 2 provides a summary of the magnitude, year, event name, as well as the station name and owner for each record.

NO.	Earthquake			Recording Station	
	M	Year	Name	Name	Owner
Pulse Records Subset					
1	6.5	1979	Imperial Valley	El Centro Array #7	USGS
2	6.9	1980	Irperial, Italy-01	Sturno	ENEL
3	6.5	1987	Superstition Hills-02	Parachute Test Site	USGS
4	6.9	1989	Loma Prieta	Saratoge- Aloha	CDMG
5	6.7	1992	Erzican, Turkey	Erzican	...
6	7.3	1992	Landers	Lucerne	SCE
7	7.6	1999	Chi Chi, Taiwan	Tcu102	CWB
No Pulse Records Subset					
8	6.8	1985	Nahanni, Canada	Site 1	...
9	6.8	1985	Nahanni, Canada	Site 2	...
10	6.9	1989	Loma Prieta	BRAN	UCSC
11	6.9	1989	Loma Prieta	Corralitos	CDMG
12	6.7	1994	Northridge-01	La- Sepulveda VA	USGS-VA
13	7.6	1999	Chi Chi, Taiwan	Tcu067	CWB
14	7.6	1999	Chi Chi, Taiwan	Tcu084	CWB

Table 2. Earthquake Event and Recording Station Data for the Near-Field Record Set.

Building designs and simulation model

the building design consists of a 20-story reinforced concrete (RC) moment frame with a shear wall structure. The design follows the provisions outlined in the Code for Design of Concrete Structures (ACI-318) and the Code for Seismic Design of Structures (ASCE07-10). The RC structure has been designed with the average ductility capacity. Figure 2 presents the plan layout of the structure, with each story having 3.6 meters. In terms of materials, the shear walls and columns are constructed using concrete with a strength grade of C40. The plates and beams utilize concrete with a strength grade of C30. The reinforcing steel has a yield strength of 400 Mpa. Load information for the structure is as follows: the floor dead load is 5.0 kN/m², and live load is 2.0 kN/m². The Roof dead load is 7.5 kN/m², and the live load is 0.5 kN/m². Table 3 provides additional seismic design information.

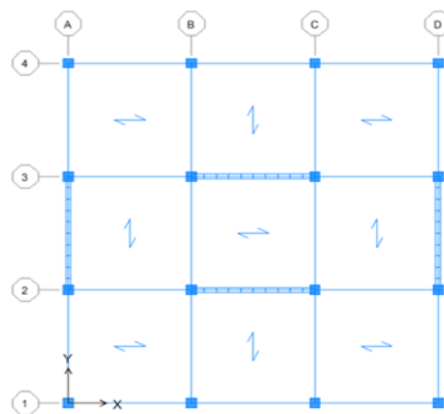


Figure 2. plan layout of the structure.

Seismic Design Category	Site Class	S _s	S ₁	F _a	F _v	SD _s	SD ₁
D	C	1.5	0.6	1	1.3	1	0.52

Table 3. Seismic design information.

For the probabilistic assessment of seismic collapse risk, robust analytical models capable of simulating the structure’s principal failure modes are essential. The analyses in this study utilize a two-dimensional model developed and implemented in OpenSees, a software program for structural analysis. The seismic response of reinforced concrete moment frames in this study is modeled using nonlinear element representations of beams, columns, and beam-column joints. Specifically, the reinforced concrete special moment frame structures utilize lumped plasticity concentrated hinge models. These models employ a material model developed by Ibarra, Medina, and Krawinkler (2005) and implemented in OpenSees. The chosen material model is capable of capturing the essential deterioration modes that lead to the sideways collapse of reinforced concrete frames. It allows for versatile modeling of cyclic behavior, as demonstrated by the tri-linear monotonic backbone curve and associated hysteretic rules shown in Figure 3. The model’s post-peak response is crucial for simulating structural collapse, as it enables the modeling of concrete crushing, rebar buckling, fracture, and bond failure through strain softening action.

To achieve this, the ModIMKPeakOriented material is utilized in OpenSees. This material represents the modified Ibarra-Medina-Krawinkler deterioration model with a peak-oriented hysteretic response. It’s worth noting that the calibration of this material’s hysteretic response was conducted using 200 experimental data of RC beams to estimate the deterioration parameters of the model. In the analysis, beams and columns are modeled using the lumped plasticity approach, also known as the plastic hinge approach. The hysteretic model developed by Ibarra-Medina-Krawinkler is employed for its ability to simulate the strength and stiffness degradation experienced during seismic collapse.

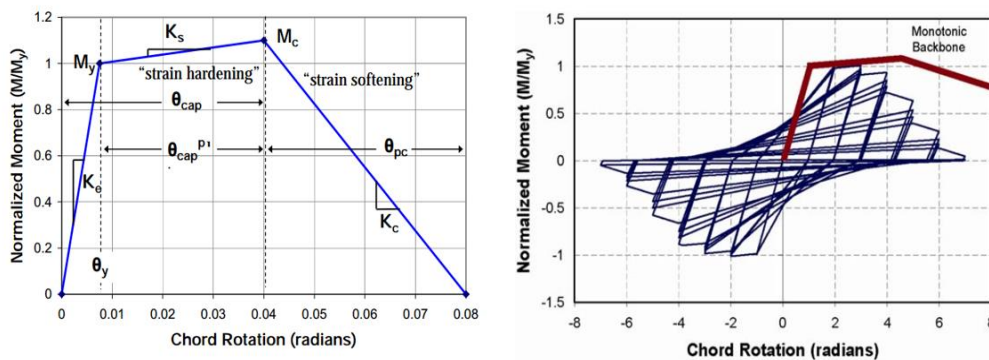


Figure 3. Monotonic and cyclic behaviour of the component model used to model reinforced concrete beam-column elements.

Code-conforming reinforced concrete frame systems in new tall buildings are designed to have joints that can effectively develop the flexural strength of the beams connected to the joint. Since a significant portion of the inelastic response of reinforced concrete beams and columns occurs in the beam-column joints, modeling the inelastic behavior of the beams, columns, and joints is interconnected. One way to represent a reinforced concrete beam-column joint region is illustrated in Figure 4. It utilizes five inelastic springs, coupled through kinematic constraints, to capture the finite size of the joint. In this representation, each of the four springs connecting the joint to the adjacent beam or column is shared by both the joint and the connecting element. These springs are calibrated to account for inelastic deformations in the member’s plastic hinge as well as the bond-slip and yield penetration into the joint. They are identical to the springs at each end of the flexural member shown in Figure 3. Additionally, there is a central spring within the joint that models the panel deformations resulting from significant shear force transfer through the joint. It is crucial to differentiate and consider deformations associated with the flexural response of the member, anchorage into the joint, and joint panel shear when calibrating flexural hinges for reinforced concrete frame components. To achieve this, the Join2D element is employed in OpenSees. This element allows for the construction of a two-dimensional beam-

column-joint object, representing the joint as a parallelogram-shaped shear panel. The adjacent elements are connected to the midpoint of this panel, referred to as external nodes. These external nodes serve as the connection points between the joint element and the surrounding structure during analysis.

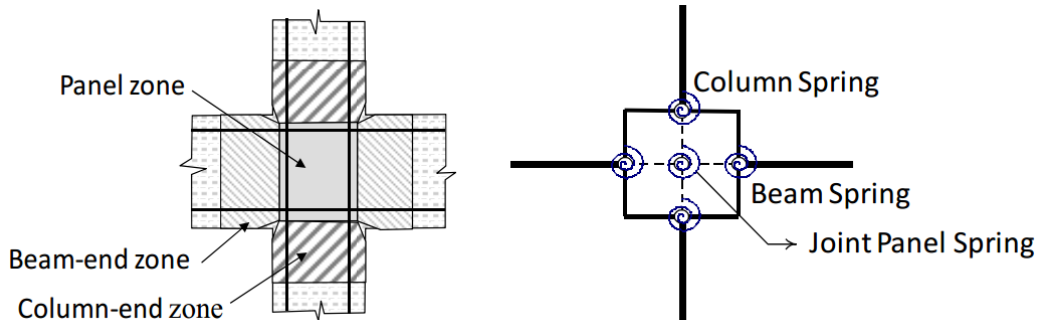


Figure 4. The idealization of reinforced concrete beam-column joint.

In this study, the fiber Beam-Column model has been used to the modeling of a shear wall. Fiber (distributed inelasticity) beam-column models involve subdividing the wall section into concrete and steel fibers. Fiber models have become increasingly common in practice as they have been implemented in commercially available analytical software. They address many of the shortcomings observed in equivalent beam-column models. In a fiber model, the cross-section geometry was prescribed, and concrete and steel fibers are individually defined.

Collapse assessment approach

To account for the inherent variability in earthquakes when analyzing the seismic response of a structure, an incremental dynamic analysis (IDA) method has been developed. The seismic collapse risk of the building was evaluated using IDA. In IDA, the analytical model of the building is subjected to a specific earthquake time history, and the response of the structure, such as story displacements and floor accelerations, is tracked. The input ground motion is scaled to progressively higher levels of intensity until collapse occurs, which is indicated by uncontrollable inter-story drifts. This iterative process is repeated for all earthquake records and each structure.

In this study, the ground motion intensity is scaled based on the spectral acceleration at the fundamental period of each building, ($S_a(T_1)$). Statistics from the incremental dynamic analysis (IDA) results are used to derive a fragility curve. The Hunt & Fill algorithm has been utilized for IDA in this study. initially, a small value of the intensity index is chosen to ensure that the structure remains within the linear range. (In the case of $S_a(T_1)$, a value of 0.05 is deemed adequate). It is considered as the initial value of the intensity index. The hunting stage aims to determine the collapse intensity index using the minimum number of points. During this stage, the intensity index incrementally increases by a small value (e.g., 0.05) with each step. This gradual increase allows the intensity values to leap to the point where collapse occurs. To achieve accurate collapse intensity, it is necessary to backtrack slightly from the leaping values. Several steps are taken using the previous point to determine the intensity index corresponding to the collapse conditions (end of the hunt). This approach improves the accuracy of IDA, particularly in the nonlinear graph region. Once the collapse intensity index is obtained, which is a major objective of the incremental dynamic analysis method, additional points are assigned to each record to increase precision in the initial part of the graph. This stage is known as "fill," where extra points are added at intervals based on the initial points. The collapse fragility function is defined using the median and standard deviation of the ground motion intensities at which collapse occurs in IDA, assuming a lognormal distribution.

The lognormal standard deviation (β) provides a direct measure of the dispersion in the analysis results. Variability in ground motion characteristics and structural modeling introduces uncertainties in predicting structural collapse. Record-to-record variability arises from differences in ground motion frequency content and other geological characteristics, and it is quantified by the dispersion computed from the incremental dynamic analysis (IDA) results. Modeling

uncertainty reflects the degree to which the simulation model accurately represents the physical properties and seismic response of a structure, and it is particularly crucial for predicting structural collapse. This type of uncertainty arises from nonlinear modeling decisions and the definition of building model parameters. Various methods have been employed to account for the effects of modeling uncertainty on seismic fragility curves.

In the analysis results presented here, consideration has been given to both structural design variability and experimental data variability. This is done to examine how engineering regulations, which often enforce conservative limitations, shape the structural design beyond the minimum standards. Uncertainty in experimental data arises because obtaining the actual behavior of structures, especially the nonlinear response of systems unaffected by severe earthquakes, is not solely achievable through analytical models. Therefore, it is necessary to provide sufficient experimental data to develop models that accurately represent the behavior of the structure. The total variability can be determined by taking the square root of the sum of the squares.

$$\beta_{TOT} = \sqrt{(\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{MDL}^2 + \beta_{TD}^2)} \quad (2)$$

In terms of record uncertainties, the β_{TOT} uncertainty encompasses the whole, while β_{RTR} , β_{DR} , β_{MDL} , β_{TD} , represent the uncertainties associating with records, design, modelling and experimental factors, respectively. Figure 5-6 displays the incremental dynamic analysis (IDA) results for a 20-story RC building with shear walls subjected to a range of near and far-field ground motions. Statistical analysis of the IDA results is utilized to derive a fragility curve, as depicted in Figure 7, which represents the probability of collapse as a function of spectral intensity. The collapse fragility function is defined by the median and standard deviation of ground motion intensities at which failure occurs in the IDA, assuming a lognormal distribution. The lognormal standard deviation (β) provides a direct measure of the dispersion in the analysis results.

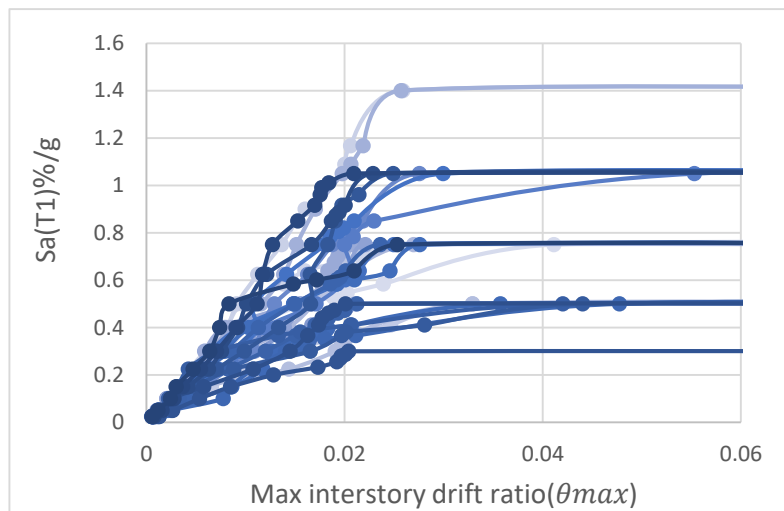


Figure 5. IDA curve for far-field records.

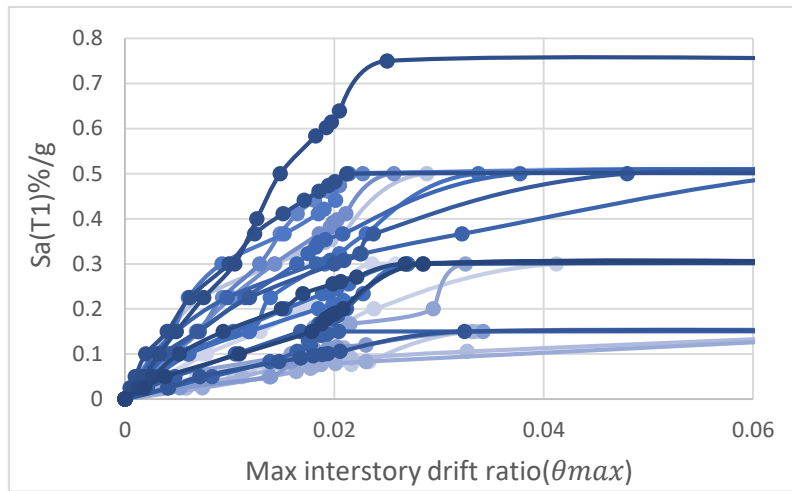


Figure 6. IDA curve for near-field records.

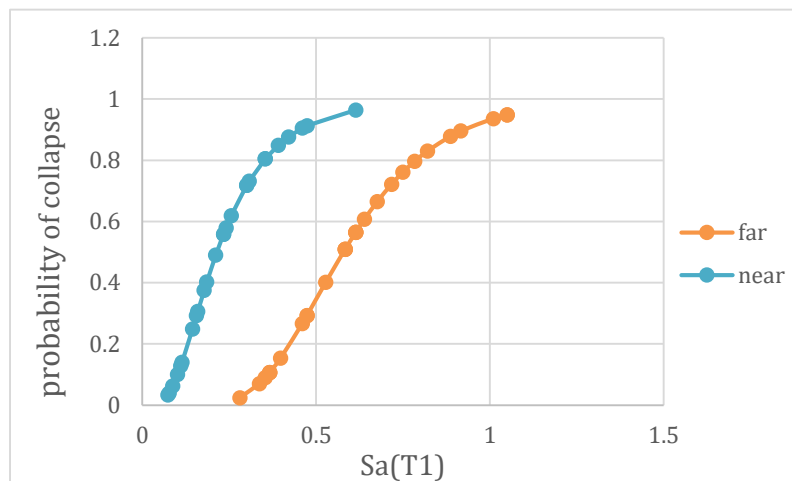


Figure 7. Comparison of fragility curves

The risk of collapse and the associated factors for near and far-field faults, which were obtained from the MATLAB program, are presented in Table 4 along with the results of uniform acceleration calculations.

20 story						
field	Sa(uniform hazard)(g)	Sa(risk target ground motion)(g)	risk coefficient	probability of collapse in 1 year	probability of collapse in 1 year(%)	probability of collapse in 50 years (%)
far	0.623	1.34	2.15	6.97E-04	6.97E-02	3.429
near	0.623	0.478	0.767	2.37E-03	2.37E-01	11.182

Table 4. the possibility of collapse and risk factors for near and far field fault.

Conclusions

With the development of the economy, high-rise structures continue to emerge, and the design for collapse resistance of such structures has gained increasing attention. However, due to the complexity of collapse and the lack of a complete technical standard system, current specifications and design criteria are still evolving. In this study, we have proposed seismic collapse analysis methods and integral iteration and solution methods, which have proven effective in obtaining seismic collapse analysis data for structures. Despite the shift towards a risk-targeted approach, the new seismic hazard maps do not explicitly consider the collapse risk in the near-fault region. The MCE (Maximum Considered Earthquake) intensity in these regions still relies on deterministic values. Our findings suggest that the collapse risk of RC moment frame with shear wall buildings in these regions may be significantly higher than the 1% probability of collapse in the 50-year target set for near and far-field sites. There is a notable difference in the probability of collapse between the near and far-field regions. Therefore, using the probability of a 1% collapse in 50 years to determine the MCE acceleration for all buildings, as suggested by this study, may not be appropriate.

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