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Seismic vulnerability of non-structural members in reinforced concrete buildings located in Tehran

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Abstract:

Reinforced Concrete (RC) buildings are a common type of structure. Dual systems (containing RC shear walls and moment resisting frame), and moment resisting frame systems are the most common types of RC buildings in Iran. Some researchers have studied the seismic reliability of bridge structures using field data. However, in Iran, real field data is not used to analyze the reliability of RC buildings. In this study, reliability analysis is used to assess the failure of non-structural members in the RC buildings. The probability distribution of the concrete and steel bars strength is gathered by using field tests. The tests were done in 110 RC buildings in Tehran. Afterward, a series of time history analysis were done to determine the probability of failure in non-structural members. Monte Carlo sampling is used for reliability analysis. The reliability of two common RC structural systems are compared under different earthquake records. It is found that the dual system can have a better performance under seismic excitation and it can reduce the damage in an earthquake.

1. Introduction

Reinforced Concrete (RC) buildings are one of the most common types of structures in Iran. RC moment resisting frame system and dual system (including shear walls and moment resisting frames) are vastly used for commercial, industrial, office, and residential buildings. So, a significant financial investment has been dedicated to RC buildings and it is important to quantify seismic vulnerability of these buildings. On the other hand, some earthquake events have caused severe damage to RC buildings (e.g. 1994 Northridge earthquake in USA, 1999 Kocaeli earthquake in Turkey, 2003 Bam earthquake in Iran) [1].

It is obvious that a lot of uncertainties exist in the seismic excitation and structural capacity. Therefore, probabilistic approach should be used for evaluating seismic performance of structures [1]. Reliability analysis is the most suitable approach for evaluating the effectiveness of a structural system against earthquakes [2]. The reliability of structures provides tools which makes it possible to quantify the uncertainties and assess the vulnerability of the structures [3].

In recent years, extensive research has been done to evaluate the performance of RC buildings against earthquakes [4-11]. Thinley and Hao (2017) studied seismic performance of RC buildings in Bhutan based on fuzzy probability analysis [12]. Haeri Kermani and Fadaee (2013) studied seismic vulnerability of RC buildings using a vector intensity measure [1]. Lynch et al. (2011) studied seismic performance of RC frame buildings in southern California [13]. Çavdar et al. (2018) studied earthquake performance of RC shear-wall structure using nonlinear methods [14]. Kitayama and Constantinou (2019) studied probabilistic seismic performance of seismically isolated buildings

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designed by the procedures of ASCE/SEI 7 and other enhanced criteria [15]. Moreover, some researchers have investigated the vulnerability of existing buildings under seismic loads. Hancilar et al. determined the vulnerability of existing school buildings in Turkey. In their research, the vulnerability was determined through fragility curves, and the probability of failure was determined for different levels of performance [16].

On the other hand, in some special types of structures, the seismic reliability was determined by using field data. For RC structures, field data can be gathered by using rebound hammer or ultrasonic tests. Huang et al. studied the seismic reliability of RC bridge structures under earthquake by using non-destructive tests [17]. Küttenbaum et al. studied the reliability of constructed bridges bases on field data [18].

In the RC building type structures, extensive studies have been carried out to assess the seismic reliability. However, very little attention has been paid to the field data. Moreover, in the past decades, several earthquakes have occurred in Iran, where the number of fatalities was not large in some of the earthquakes; however, drastic damage was reported in the non-structural members. Some of the damage in the non-structural members reported from Bojnord earthquake which occurred in May 2017, is shown in Figure 1.

In this research, the vulnerability of non-structural members in RC buildings is investigated. The reliability analysis is used for this study. It is assumed that the concrete compressive strength and, steel bars tensile strength are the main variables. Rebound hammer test is used to determine the concrete compressive strength properties. In addition, tensile test is used to determine the yielding strength of the bars. Finally, the fragility curves are derived for different values of Peak Ground Acceleration (PGA).

2. Filed data

In this study, the compressive strength of the concrete and yielding strength of the steel bars are assumed to be main variables. Some researchers used the Rebound Hammer, which is a well-known non-destructive test used for measuring the compressive strength of the concrete. Huang et al. used the rebound hammer to measure the compressive strength of the concrete in bridge structures and for adaptive reliability analysis [17]. The specifications of the rebound hammer test were published in ASTM C805 standard [19]. According to ASTM C805 standard, some information should be reported while the rebound hammer test is being done. This information is listed in Table 1 and Table 2. In Table 1, the f'c denotes the compressive strength of the cylindrical specimen. The cylindrical specimens of concrete have a diameter of 15 cm and height of 30 cm. Moreover, the average of the rebound number is five in each point. In addition, in Figure 2, the devices of non-destructive tests were shown. The brand of the rebound hammer is NOVOTEST. Note that before conducting each rebound hammer test, the rebar scanner was used in order to identify the cover of rebar in the concrete. The device for rebar scan is ZBL-R660.

Fig. 1: Non-structural members damage in the Bojnord 2017 earthquake. The pictures were taken by the engineers of the Mandegar Structures Q.C. & Inspection Company.

a) b)

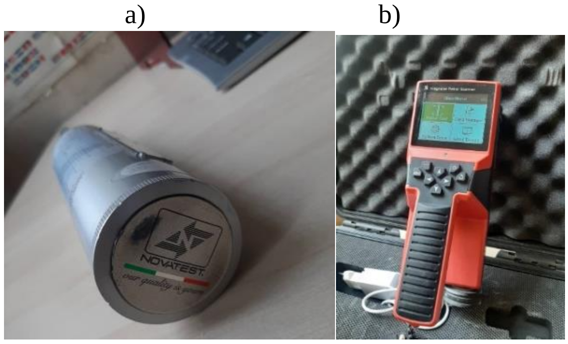

Fig. 2: The pictures of the devices used by Mandegar Structures Q.C. & Inspection Company for non-destructive tests: a) hammer, b) rebar scanner

In this study, the rebound hammer test was conducted for 110 RC buildings in Tehran. The RC buildings had moment resisting frames or dual (including moment resisting frame and shear wall) systems. The non-destructive tests were conducted for the beams, columns, and shear walls. In Table

3, the number of rebound hammer tests for each structural member is listed.

Afterward, according to non-destructive tests, the probability of distribution for the f'c in beams, columns, and walls is expressed in Table 4. Note that these probabilities of distribution are derived by Matlab 8.3.0 Distribution Fitter toolbox. It is observed that the average strengths and coefficients of variation are very close to each other in the beams and columns. Moreover, in the beams and columns, the average strength of concrete is about 10% less than the strength of the cylindrical specimen. However, in the shear walls, the average compressive strength of the concrete is very near to the strength of the cylindrical specimen. In addition, the coefficient of variation in the shear walls is about 60% more than the beams and columns.

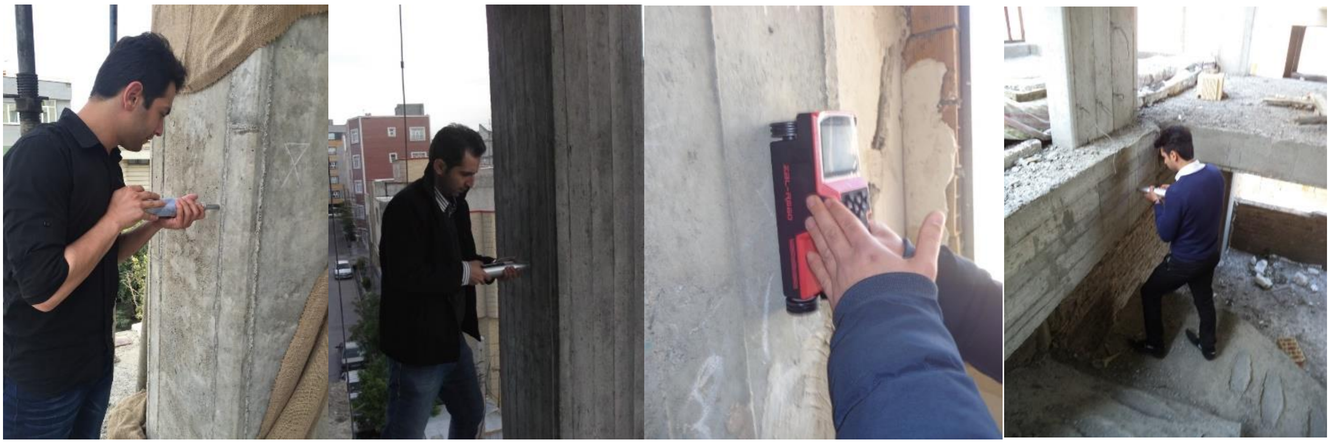

Fig. 1: Sample pictures of non-destructive tests by the engineers of Mandegar Structures Q.C. & Inspection Company

Table. 1: Information of rebound hammer test

Date	Temperature (C^0)	Time of test	Age of concrete	Structural members dimensions	f'_c
Winter of 2020 and 2019	8-15	Between 10 AM to 2 PM	28-180 days	Beam: 30 to 60 cm Column: 40 to 70 cm Wall: 30<thickness<40 cm	30 MPa

Table. 2: Information of rebound hammer test

Concrete surface characteristics	Surface moisture condition	The angle of hammer with horizontal axis	Date of hammer calibration	Type of the form material	Curing condition
Formed	Dry	0^0	January 10th, 2019 January 10th, 2020	Steel	Wet covering for one week

Table. 3: number of conducted tests in structural members

Number of rebound hammer tests in beams	Number of rebound hammer tests in columns	Number of rebound hammer tests in shear walls
2000	2000	2000

The yielding strength of the steel bars are determined by tensile test. The tensile test was done in the laboratory of civil engineering department located in Iran University of Science and Technology. In Iran, the most common type of steel bar is AIII. The bar type AIII, has a yielding strength of 400 MPa. The bars with 10, 12 and 14 mm diameters are used for transverse reinforcement, and the bars with a diameter higher than 14 mm are used for longitudinal reinforcement. The yielding stress of the bars are listed in Table. **5**. Note that in this table, the engineering stress is listed. Again, the probability distribution is derived by using Matlab 8.3.0 Distribution Fitter toolbox. It is seen that all of the average yielding stresses are higher than 400 MPa. Moreover, the steel bar with a diameter of 10 mm has the highest coefficient of variation and the bars with 12, 16, 18, 20, and 22 mm diameter have smaller coefficient of variation than the other bars.

3. Non-linear dynamic time history analysis

For nonlinear time history analysis, a six-story RC building is considered. The building has a plan as shown in Fig. **2**. The building is symmetric in two orthogonal directions. Moreover it was designed using both the moment resisting frame and dual systems. The cross sectional properties of the shear walls, beams, and columns are listed in Table. **6** and the rebar percentage of the beams are stated in Table. **7**. The buildings were designed according to Iranian 2800 standard and Iranian guide for design of RC structures. Note that the 4th version of the 2800 standard was used for design. The height of each story is 3.2 m. It is assumed that the building is located in Tehran and the soil type is III according to Iranian 2800 seismic code. In each floor the dead load is assumed to be 500 kg/m^2 and live load is 200 kg/m^2. The compressive strength of concrete is 30 MPa and the yield strength of the bars is 400 MPa. Note that, the mentioned strength is the nominal strength of the concrete and steel bars. In the reliability analysis, these parameters will be selected according to Table. **5**.

Table. 4: The probability of distribution properties for *f'c* in beams, columns, and walls

Type of member	Type of distribution	Average (MPa)	Coefficient of variation
Beam	Normal	27.27	0.278
Column	Normal	27.51	0.252
Wall	Normal	30.08	0.403

Table. 5: Probability distribution of the tested bars

Bar diameter (*mm*)	Symbol	Type of fitted distribution	Mean (*kgr/cm^2*)	Coefficient of variation	Number of specimen
10	Φ10	Normal	430	0.198	400
12	Φ12	Normal	470	0.066	200
14	Φ14	Normal	469	0.100	400
16	Φ16	Normal	488	0.065	400
18	Φ18	Normal	504	0.082	500
20	Φ20	Normal	506	0.063	500
22	Φ22	Normal	496	0.046	300
25	Φ25	Normal	471	0.126	200

In this study, the non-linear structural analysis was performed by using OpenSees. In the OpenSees, the materials Concrete02 and Steel02 were used to model the concrete and steel bars [20]. The elasticity modulus of concrete is derived by [21]:

$$E_c = (3300\sqrt{f'_c} + 6900)(\frac{\gamma_c}{23})^{1.5} \quad (1)$$

where γ_c is the special weight of the concrete. According to Iranian 2800 seismic standard, the dominant frequency of the soil type III is between 0.5 and 1.5 Hz. Therefore three earthquake records are selected in a way that the dominant frequency of the records is between 0.5 to 1.5 Hz. Fast Fourier Transform (FFT) is used to determine frequency content of the earthquake records. The selected earthquake records are San-Fernando (1971), Loma-Prieta (1995), and Kobe (1989) [22]. The accelerogram of the records are shown in Figure 5. Moreover the FFT of the selected earthquake records are shown in Figure 6. It is observed that the dominant frequency of these records is between 0.5 and 1.5 Hz. It is seen that the San-Fernando earthquake has a wider range of frequency content. In addition, the spectral acceleration of the earthquake records are shown in Figure 7. In Figure 7, the maximum acceleration of the records is 0.1g. Note that the earthquakes are applied in the X direction.

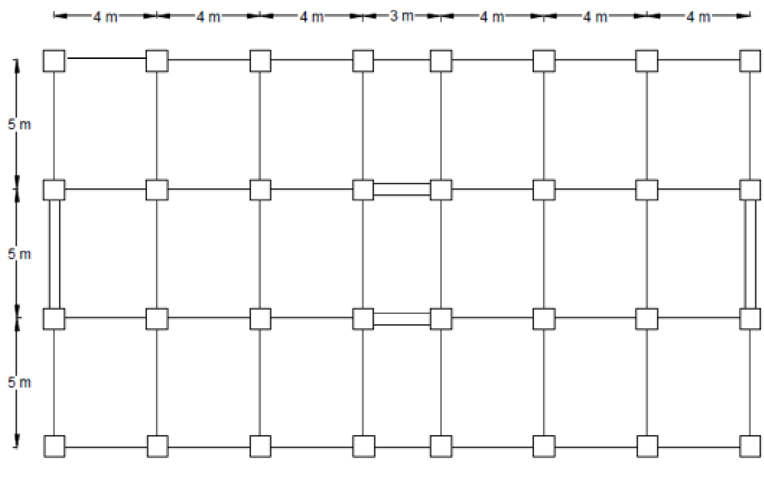

Fig. 2: the plan of modeled RC buildings

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Table. 6: Natural period and structural members' properties in modeled buildings

Structural system	First natural mode period	Floor number	Shear wall thickness	Column Section properties	Beam dimensions (cm)
Moment resisting frame	0.88 sec	1,2	-	50x50 cm 2.0 % reinforcement	50x50
		3,4	-	50x50 cm 1.6% reinforcement	40x40
		5		40x40 1.6% reinforcement	40x40
		6	-	35x35 1.5% reinforcement	35x35
Shear wall	0.57 sec	1,2	35 cm	50x50 1.5% reinforcement	40x40
		3,4	35 cm	40x40 1.2% reinforcement	40x40
		5	35 cm	35x35 1.5% reinforcement	35x35
		6	35 cm	35x35 1% reinforcement	30x30

4. Reliability analysis

Herein, the Monte Carlo method is used for reliability analysis. In the Monte Carlo Simulation (MCS) method, the Boolean function is defined as [23]:

$$I(x) = \begin{Bmatrix} 1 \quad if \bigcup_{k=1}^{N_{CS}} \bigcap_{i \in C_k} g_i(x) < 0 \\ 0 \quad Otherwise \end{Bmatrix} \tag{2}$$

In above equation, I(x) can be equal to zero or 1. When I(x)=1, the failure has occurred, and when I(x)=0, the failure has not taken place. In the equation **Error! Reference source not found.**, g(x)<0 means the failure has occurred.

In this study, it is assumed that if maximum inter-story drift is more than 0.005 story height, the failure has occurred. Note that, according to Iranian 2800 seismic standard, the allowable inter-story drift for non-structural members failure is 0.005 height of the story. In MCS method, the probability of failure can be calculated by [23]:

$$P_f = \frac{1}{N_s} \sum_{i=1}^{N_s} I(x_i) \tag{3}$$

In above equations, Ns denotes the number of samples.
For numerical analysis, the OpenSees is connected to the RT software. The sampling process is done by RT, and in each step the materials properties were produced in RT. Moreover the time history analysis was done by OpenSees.

Table. 7: Rebar percentage of the beams

Floor number	Structural system	Beam dimensions (cm)	1/3 Middle at top	1/3 Middle at bottom	1/3 beginning and end at Top	1/3 beginning and end at bottom
1,2	Moment resisting frame	50x50	0.5%	0.5%	1%	1%
3,4		40x40	0.6%	0.6%	1.2%	1.2%
5		40x40	0.4%	0.4%	0.8%	0.8%
6		35x35	0.6%	0.6%	1.2%	1.2%
1,2	Shear wall	40x40	0.4%	0.4%	0.8%	0.8%
3,4		40x40	0.35%	0.35%	0.7%	0.7%
5		35x35	0.4%	0.4%	0.8%	0.8%
6		30x30	0.4%	0.4%	0.8%	0.8%

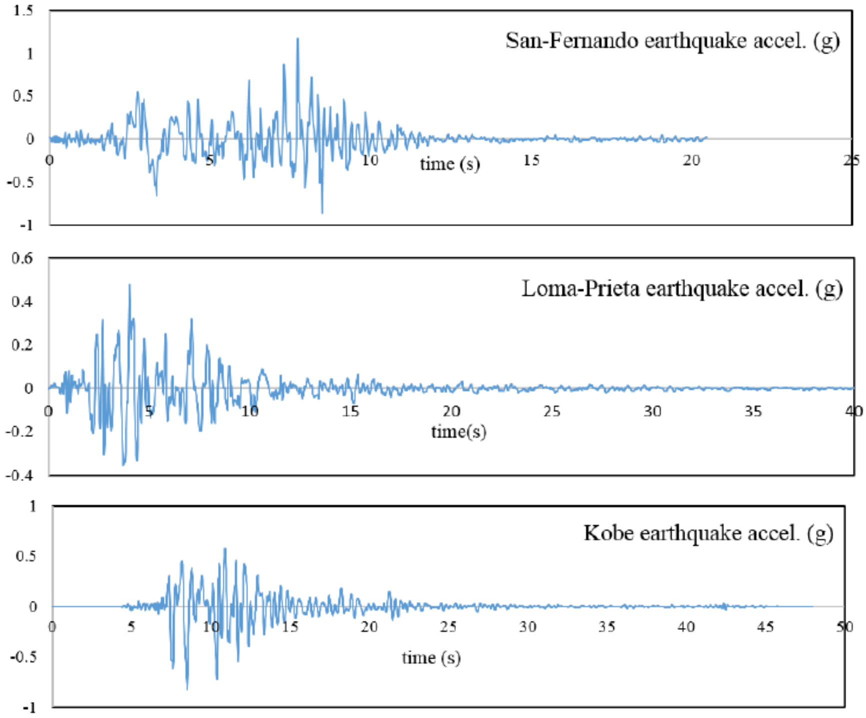


Fig. 3: The accelerogram of the selected earthquakes

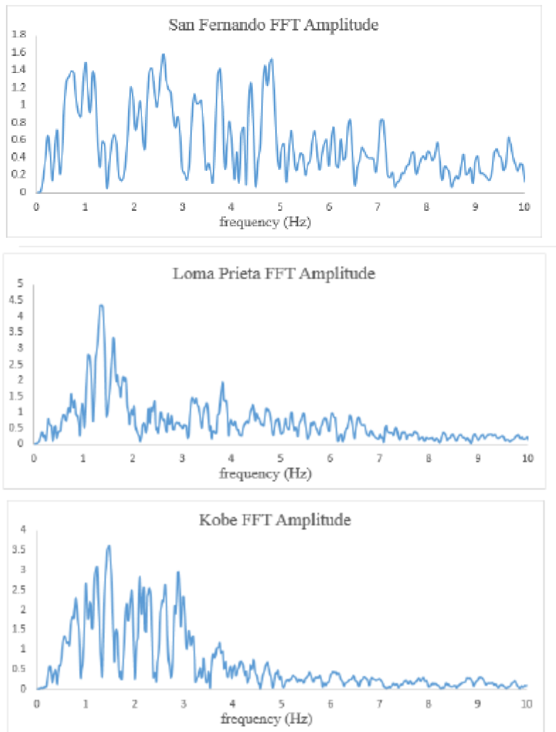


Fig. 4: FFT of the selected earthquakes

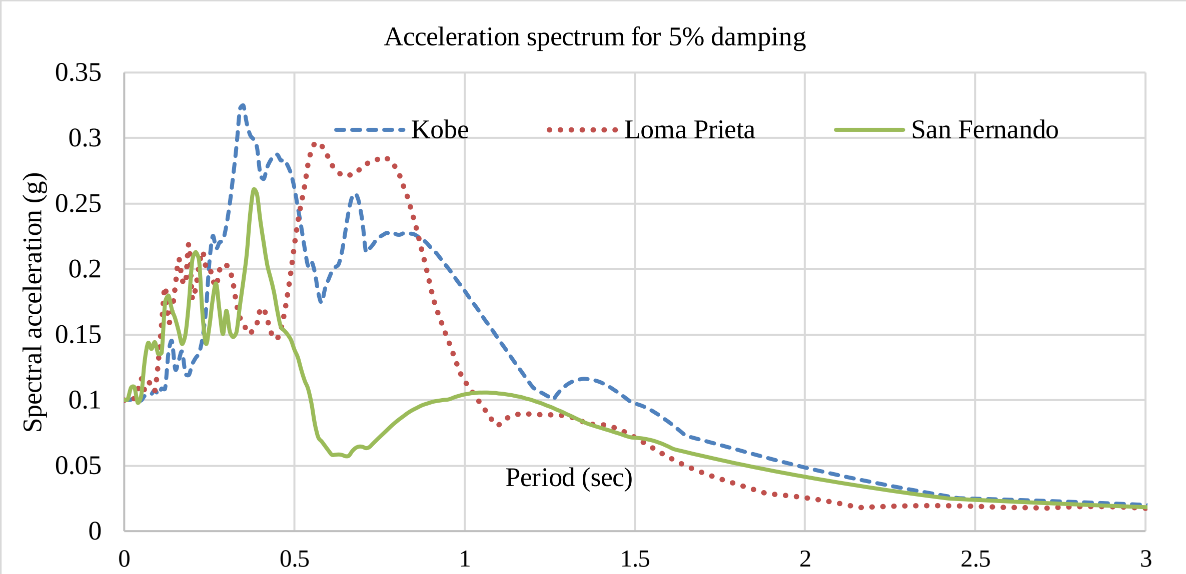


Fig. 5: Spectral acceleration of the earthquake records

5. Numerical Results

In this section, the results of the reliability analysis is presented. Before presenting the fragility curves, the displacement response of two systems are compared. In Fig. **6**, the displacement of the 3rd and 6th story for both systems is shown. The selected earthquake record is Kobe. The record is scaled and the maximum acceleration is selected to be 0.2g. In both of the systems, the maximum inter-story drift exceeds 0.005h. Therefore, in both structures the non-structural members were damaged. Moreover, both systems have a small amount of residual displacement, which means that some of the structural elements have entered the non-linear region. The residual displacement is smaller in the building with moment resisting frame system.

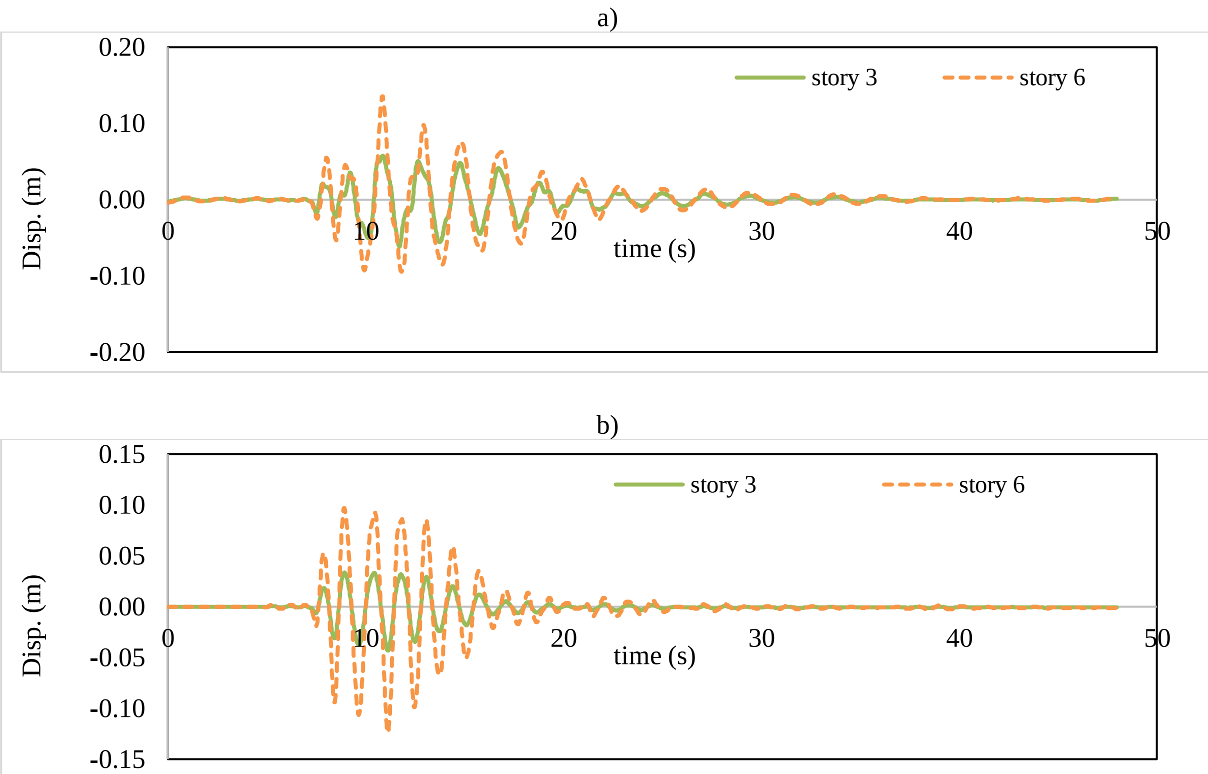


Fig. 6: The displacement of the 3rd and 6th story a) building with moment resisting frame system b) building with dual system

The fragility curves of the modeled buildings are shown in Fig. 7. Likewise, it is mentioned that the curves are plotted for the failure of non-structural elements. It is observed that the building with dual system has a better performance under all of the earthquake records. Using dual system instead of moment resisting frame can increase the maximum acceleration of failure in non-structural members between 10% and 25%. The minimum PGA for non-structural members' failure is 0.1g, which occurred in the building with moment resisting frame in Kobe earthquake. For the building with moment resisting frame, the minimum failure PGA is between 0.1g and 0.136g. Moreover, for moment resisting frame the maximum failure acceleration is between 0.12g and 0.168g. For the building with dual system, the minimum acceleration of failure is between 0.11g and 0.16g and the maximum acceleration of failure is between 0.14g and 0.21g. Better performance of the dual system can occur for two reasons. First, as mentioned in the last sections, the average concrete strength in the shear walls is more than the moments resisting frame. Second, the stiffness of the dual system is more than the moment resisting frame system.

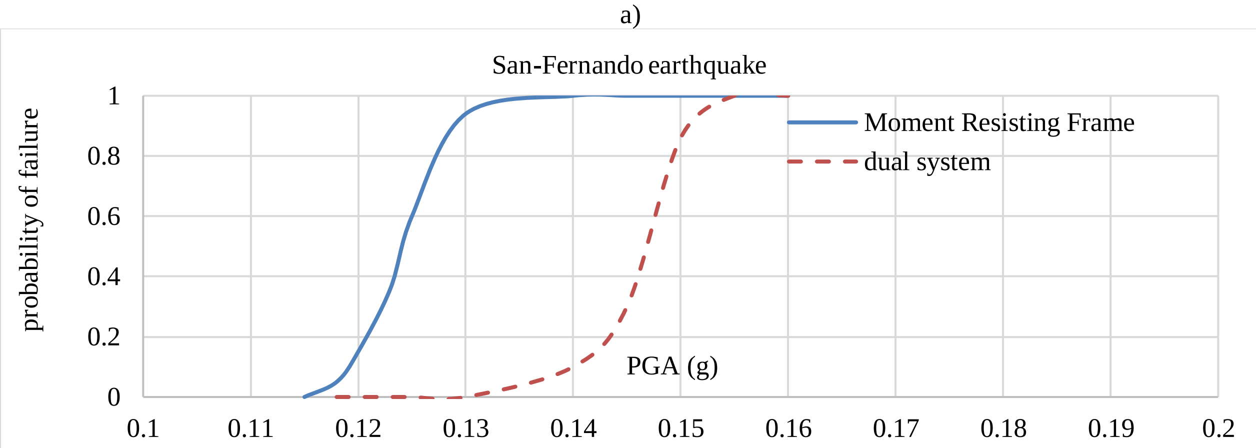
5. Conclusions

In this study the field data from constructed RC buildings were used for reliability analysis of RC buildings. The reliability analysis was done for RC moment resisting frame and dual systems. Fragility curves were derived for reliability analysis.

The field data was gathered for concrete and steel bars. The concrete compressive strength and bar tensile strength were the proposed field data. The compressive strength of concrete was measured by rebound hammer and the yielding strength of the bars was measured by bar tensile test. It was found that the concrete quality in the shear walls is better than the beams and columns. The average concrete strength in the shear wall was a little more than the nominal concrete strength. However the average concrete compressive strength in the beams and columns was less than the nominal concrete compressive strength. For the steel bars with different diameters, the yielding strength was more than the nominal yielding stress.

The reliability analysis was done by deriving fragility curves for non-structural members. Two six-story RC buildings were designed according to Iranian 2800 standard and Iranian concrete code. One of the buildings had moment resisting frame and the other one had dual system. The buildings were located on soil type III according to Iranian 2800 standard. Three earthquake records were selected. The failure criterion was selected by using inter-story drift. According to the results, using the dual system can increase the reliability of non-structural members in RC buildings under earthquake loading.

a)



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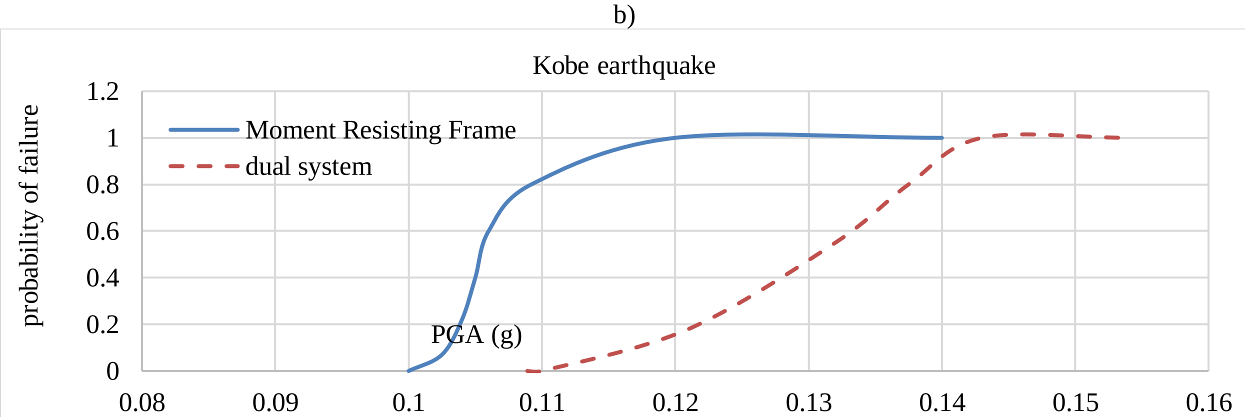

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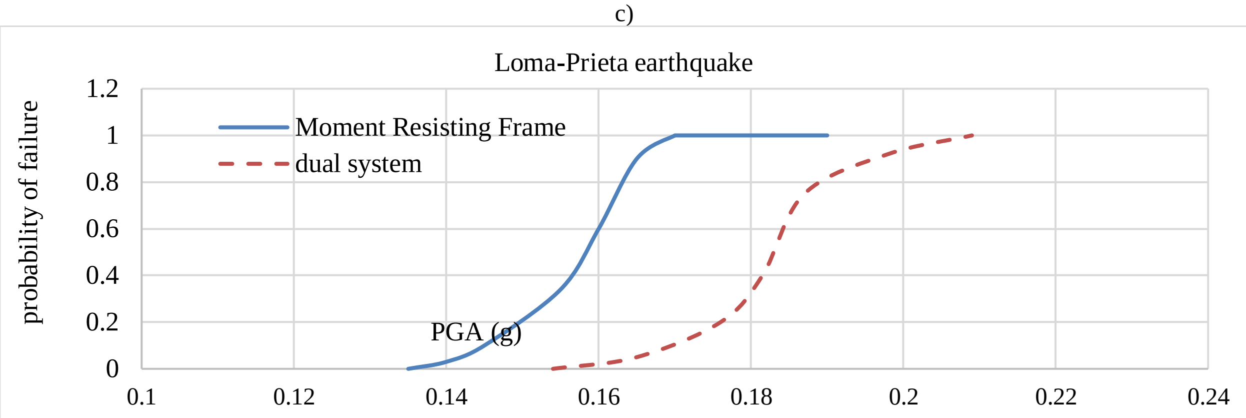

Fig. 7: Fragility curves of the building systems under earthquake a) San-Fernando earthquake b) Kobe earthquake c) Loma-Prieta earthquake

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