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

Aliasghar Amir Kardoust*, Seyyed Azim Hosseini**, Hamidreza Rabeifard***, Seyyed Mohammad Seyyed Hosseini****, Abbas Akbarpour NickghalbRashti*****

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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 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]. Küttinbaum 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]. According to ASTM C805 standard [19], 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.

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

| Date                  | Temperature (°C) | 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                        | $\theta$                               | January 10th, 2019          | 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

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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/m² and live load is 200 kg/m². 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/cm²) | Coefficient of variation | Number of specimen |
|-------------------|--------|------------------------------|----------------|--------------------------|--------------------|
| 10                | \( \Phi 10 \) | Normal                       | 430            | 0.198                    | 400                |
| 12                | \( \Phi 12 \) | Normal                       | 470            | 0.066                    | 200                |
| 14                | \( \Phi 14 \) | Normal                       | 469            | 0.100                    | 400                |
| 16                | \( \Phi 16 \) | Normal                       | 488            | 0.065                    | 400                |
| 18                | \( \Phi 18 \) | Normal                       | 504            | 0.082                    | 500                |
| 20                | \( \Phi 20 \) | Normal                       | 506            | 0.063                    | 500                |
| 22                | \( \Phi 22 \) | Normal                       | 496            | 0.046                    | 300                |
| 25                | \( \Phi 25 \) | 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]:

\[
E_c = (3300\sqrt{f'c} + 6900)(\gamma_c/23)^{1.5}
\]  

where \( \gamma_c \) 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](image-url)
### 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]:

\[
I(x) = \begin{cases} 
1 & \text{if } \bigcup_{k=1}^{Nc} \bigcap_{c_k} g_i(x) < 0 \\
0 & \text{Otherwise} \end{cases} 
\]  

(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]:

\[
P_f = \frac{1}{N_s} \sum_{i=1}^{N_s} I(x_i) 
\]  

(3)

In above equations, \( N_s \) 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          | Shear wall               | 40x40                | 0.6%              | 0.6%                 | 1.2%                         | 1.2%                            |
| 5            | Shear wall               | 40x40                | 0.4%              | 0.4%                 | 0.8%                         | 0.8%                            |
| 6            | Shear wall               | 35x35                | 0.6%              | 0.6%                 | 1.2%                         | 1.2%                            |
| 1,2          | Shear wall               | 40x40                | 0.4%              | 0.4%                 | 0.8%                         | 0.8%                            |
| 3,4          | Shear wall               | 40x40                | 0.35%             | 0.35%                | 0.7%                         | 0.7%                            |
| 5            | Shear wall               | 35x35                | 0.4%              | 0.4%                 | 0.8%                         | 0.8%                            |
| 6            | Shear wall               | 30x30                | 0.4%              | 0.4%                 | 0.8%                         | 0.8%                            |
Fig. 3: The accelerogram of the selected earthquakes

Fig. 4: FFT of the selected earthquakes
5. Numerical Results

In this section, the results of the reliability analysis are 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.

![Acceleration spectrum for 5% damping](image)

**Fig. 5:** Spectral acceleration of the earthquake records

![Disp. (m) vs. time (s) for story 3 and story 6](image)

**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.
References:

[1] Haeri Kermani A and Fadaee MJ. Assessment of seismic reliability of RC framed buildings using a vector-valued intensity measure. Asian Journal of Civil Engineering 2013; 14:17-32.

[2] Lin KC, Lin CC, Chen JY and Chang HY. Seismic reliability of steel framed buildings. Structural Safety 2010; 32:174-182.

[3] Abdelnoaﬁ EG, Abdellatif K, Mohamed B and Francesc LA. Seismic performance reliability analysis for reinforced concrete buildings. Journal of Civil Engineering and Construction Technology 2011; 2: 45-53.

[4] Celik OC and Ellingwood BR. Seismic fragilities for nonductile reinforced concrete frames—Role of aleatory and epistemic uncertainties. Structural Safety 2010; 32:1-12.

[5] Haselton CB, Liel AB, Deierlein GG, Dean BS and Chou JH. Seismic collapse safety of reinforced concrete buildings. I: Assessment of ductile moment frames. Structural Engineering 2011; 137: 481–91.

[6] Jalayer F, Iervolino I and Manfredi G. Structural modeling uncertainties and their inﬂuence on seismic assessment of existing RC structures, Structural Safety 2010 32:220-28.

[7] Mahdi T and Soltan Gharieb V. Plan irregular RC frames: comparison of pushover with nonlinear dynamic analysis, Asian Journal of Civil Engineering 2011; 12: 679-90.

[8] Mehanny SSF and El Howary HA. Assessment of RC moment frame buildings in moderate seismic zones: Evaluation of Egyptian seismic code implications and system conﬁguration effects. Engineering Structures 2010; 32:2394–406.

[9] Shafei B, Zareian F and Lignos DG. A simpliﬁed method for collapse capacity assessment of moment-resisting frame and shear wall structural systems 2011; Engineering Structures, 33:1107-16.

[10] Kadid A, Yahiaoui D and Chebili R. Behavior of reinforced concrete buildings under simultaneous horizontal and vertical ground motions, Asian Journal of Civil Engineering 2010; 11: 463-76.

[11] Lynch KP, Rowe KL and Liel AB. Seismic performance of reinforced concrete frame buildings in Southern California. Earthquake Spectra 2011; 27: 399–418.

[12] Thinley K and Hao H. Seismic performance of reinforced concrete frame buildings in Bhutan based on fuzzy probability analysis. Soil Dynamics and Earthquake Engineering 2017; 92: 604–620.

[13] Lynch KP, Rowe KL and Abbie B. Liel AB. Seismic Performance of Reinforced Concrete Frame Buildings in Southern California. Earthquake Spectra 2011; 27: 399–418.

[14] Čavdar O, Čavdar A and Bayraktar E. Earthquake Performance of Reinforced-Concrete Shear-Wall Structure Using Nonlinear Methods. Journal of Performance of Constructed Facilities 2018; 32:1-12.

[15] Kitayama S and Constantinou MC. Probabilistic seismic performance assessment of seismically isolated buildings designed by the procedures of ASCE/SEI 7 and other enhanced criteria. Engineering Structures 2019; 179: 566–582.

[16] Ufuk Hancilar U, Çakti E, Erdik M, Franco E G, Deodatis G. Earthquake vulnerability of school buildings: Probabilistic structural fragility analyses. Soil Dynamics and Earthquake Engineering 2014; 67: 169–178.

[17] Huang Q and Paolo Gardoni P and Stefan Hurlebaus. Adaptive Reliability Analysis of Reinforced Concrete Bridges Subject to Seismic Loading Using Nondestructive Testing. ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering 2015; 1:1-14.

[18] Küttenbaum S, Taffe A, Braml T, and Maack S. Reliability assessment of existing bridge constructions based on results of nondestructive Testing. MATEC Web of Conferences 199 2018; https://doi.org/10.1051/matecconf/201819906001.

[19] ASTM (American Society of Testing Materials). (2003). “Standard Test Method for Pulse Velocity Through Concrete.” C 597 – 02.

[20] McKenna F, Fenves GL, Filipou FC, Scott M, Law K, Deierlein G, et al. OpenSees, open system for earthquake engineering simulation. Pacific Earthquake Research Center (PEER), University of California, Berkeley, Stanford University and University of Washington; 2002.

[21] ACI Committee 318. (2011). “Building code requirements for structural concrete and commentary.”ACI 318-11/318R-11, American Concrete Institute, Farmington Hills, MI, 509.

[22] Center for engineering strong motion data. Strongmotioncenter.org. https://www.strongmotioncenter.org. Accessed December 20, 2020.

[23] Nikolaidis E, Ghiocel DM, Singhal S, editors. Engineering design reliability handbook. Boca Raton, FL: CRC Press; 2004.