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RELIABILITY ASSESSMENT OF REINFORCED CONCRETE BUILDINGS USING FIELD DATA IN TEHRAN

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ABSTRACT:

Reinforced Concrete (RC) structures are a prevalent type of structure. In each year a lot of RC buildings are constructed and there is a large investment on RC buildings. Dual systems (containing RC shear walls and moment resisting frame) and moment resisting frame systems are the most common type of RC buildings in Iran. Earthquake can cause severe damages to RC buildings and it is important to identify which structural system has better performance under seismic excitation. Some researchers have studied seismic reliability of the bridges structures using field data. However, real field data are not used to analyze the reliability of RC buildings. In this study the reliability analysis is used to evaluate the performance of each structural system. The probability distribution of the concrete and steel bars strength are gathered by nun-destructive tests and bar tensile tests. The mentioned tests are done in 110 RC buildings in Tehran. A series of time history analysis are done to determine the probability of failure. Monte Carlo sampling is used for reliability analysis. The reliability of two prevalent 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 damages in the earthquake.

1. INTRODUCTION

Reinforced Concrete (RC) system is one of the most common structural systems in Iran. RC moment resisting frame system and dual system (consisting of shear walls and moment resisting frames) are vastly used for commercial, industrial, and residential buildings. So there is large economic investment on RC buildings and it is important to quantify seismic vulnerability of these buildings. On the other hand some earthquake events have caused severe damages to RC buildings (e.g. 1994 Northridge earthquake in USA, 1999 Kocaeli earthquake in Turkey, 2003 Bam earthquake in Iran) (Haeri Kermani, 2013).

It is obvious that a lot of uncertainties exist in the seismic excitation and structural capacity. So a probability approach should be used for evaluating seismic performance of structures (Haeri Kermani, 2013). Reliability analysis is the most suitable approach for evaluating the effectiveness of a structural system against earthquake (Lin, 2010). Reliability techniques provides tools which makes possible to quantify the uncertainties to assess structure performance (Abdelouafi, 2011).

In the recent years, a lot of researches have been done to evaluate the performance of the RC buildings against earthquake (Celik, 2010, Haselton, 2011, Jalayer, 2010, Mahdi, 2011, Mehanny, 2010, Shafei, 2011, Kadid, 2010, Lynch, 2011). Thinley and Hao (2017) studied seismic performance of reinforced concrete buildings in Bhutan based on fuzzy probability analysis. Haeri Kermani and Fadaee (2013) studied seismic vulnerability of RC buildings using a vector intensity measure (Haeri Kermani, 2013). Lynch et al. (2011) studied the seismic performance of reinforced concrete buildings in southern California. Çavdar et al. (2018) studied the earthquake performance of reinforced concrete shear-wall structures using nonlinear methods. Kitayama and Constantinou (2019) studied seismic performance of seismically isolated buildings designed by the procedures of ASCE/SEI 7 and other enhanced criteria. On the other hand some researchers have studied the reliability of RC bridges using non-destructive tests. Huang

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et al. (2015) studied adaptive reliability analysis of RC bridges subjected to seismic loading using non-destructive testing. Küttenbaum et al. (2018) studied the reliability of existing bridge based on the results of non-destructive testing.

Rebound hammer and ultrasonic pulse velocity tests are well-known non-destructive tests for RC structures. However, the reliability analysis of building structures have not been done by using field data. In this paper, the reliability analysis of RC buildings has been done by using filed data. The concrete strength is derived by rebound hammer and ultrasonic tests. Moreover the data of the bar strength are gathered from experimental tension tests. It should be noted that in this paper the probability distribution of the bar is derived according to the bar diameter. After gathering the field data, the seismic performance of two prevalent types of structural systems are compared. Moreover the effect of changing Peak Ground Acceleration (PGA) on the structural systems reliability is studied.

2. TESTS FOR GATHERING MATERIAL DATA

In this research, the compressive strength of concrete (f'_c) is gathered by using non-destructive tests. It is important to determine f'_c accurately. For determining the probability distribution of the f'_c, rebound hammer and ultrasonic pulse velocity tests are used. These tests have been done in 110 constructed residential RC buildings in Tehran. It should be noted that non-destructive tests were done at least 28 days after concreting. The non-destructive tests were done in the beams, columns, and shear walls. The bar tension strength is determined by using tensile test. The bars are randomly selected from 110 buildings and the tensile test was done for each bar.

In Error! **Not a valid bookmark self-reference.**, *CV* denotes the coefficient of variations.

, the bars yield stress is listed. In this table the bar strength properties are sorted according to the bar diameter. It should be noted that the number of tested bars for $\Phi 18$ and $\Phi 20$ is maximum. Also the number of tested bars for $\Phi 12$ and $\Phi 25$ is minimum. For typical RC buildings in Iran, $\Phi 18$ and $\Phi 20$ are the most prevalent bar size. In Error! **Not a valid bookmark self-reference.**, *CV* denotes the coefficient of variations.

, the Log-normal property distribution type is selected for the entire bar types. The Log-normal distribution is selected in the Matlab 8.3.0 Distribution Fitter toolbox.

Table 1. Probability distribution of the tested bars

Bar diameter (*mm*)	Symbol	Mean (*MPa*)	CV	Number of specimen
10	$\Phi 10$	430	0.198	400
12	$\Phi 12$	471	0.066	200
14	$\Phi 14$	469	0.1	400
16	$\Phi 16$	489	0.065	400
18	$\Phi 18$	504	0.082	500
20	$\Phi 20$	506	0.063	500
22	$\Phi 22$	496	0.046	300
25	$\Phi 25$	471	0.126	200

For fitting property distribution, different types of distribution was selected in Matlab 8.3.0 and the Lognormal distribution was best fitted to the samples. In **Error! Not a valid bookmark self-reference.**, *CV* denotes the coefficient of variations.

In the above table, it is seen that the average strength of the all bar size is more than 469 MPa. Only the average strength of $\Phi 10$ is 430 MPa. The maximum average strength is observed for $\Phi 18$ and $\Phi 20$. It should be noted that in Iran, $\Phi 22$, $\Phi 20$, and $\Phi 18$ are the most prevalent bars for longitudinal reinforcement in the typical buildings. However $\Phi 16$ and $\Phi 25$ are sometimes used for longitudinal reinforcement. Moreover, $\Phi 10$, $\Phi 12$, and $\Phi 14$ are usually used for the lateral reinforcement. The nominal strength of the modeled bars in Error! **Not a valid bookmark self-reference.**, *CV* denotes the coefficient of variations.

, is 400 MPa. But it is seen that the average strength of these bars is more than 400 MPa.

The probability distribution of f'_c is determined by conducting the non-destructive tests on the shear walls, beams and columns. In all of the inspected buildings, the nominal compressive strength of the concrete was 35 MPa. In

Table 2, the probability distribution of the f'_c is listed. The parameter f'_c is determined for the beams, columns and shear walls. In

Table 2 it is seen that average of f'_c, for all of the beams and columns is less than the design compressive strength of the concrete. The average of the f'_c in the beams and columns is very close to each other. But in the shear wall the average of the f'_c is higher in the shear walls. Also the variance of the f'_c in the shear walls is much larger.

Table 2. Selected probability distribution for f'_c

Structural member	Mean (MPa)	Coefficient of variation	Number of specimen
Beam	31.8	0.278	1000
Column	32.1	0.252	1000
Shear wall	35.1	0.403	1000

3. NON-LINEAR DYNAMIC ANALYSIS

Here, for the non-linear dynamic analysis, two 6-story buildings are considered. The height of each story is 3*m*. The typical plan of the stories is shown in Figure 1. The modeled building are designed according to ACI 318-05 code. It is assumed that the buildings are located in California and the soil under the buildings is type D (USGS). The section of beams and columns is the rectangular type. The floors are designed by 15 cm slabs and the slabs are modeled by rigid diaphragms. The compressive strength of the concrete is 40 *MPa* and the yield strength of the bars is 285 MPa. Other properties of the modeled buildings like section properties are listed in the Table 3 (Verderame, 2010). In the Table 3, T_1 denotes the period of 1st natural mode of the modeled buildings.

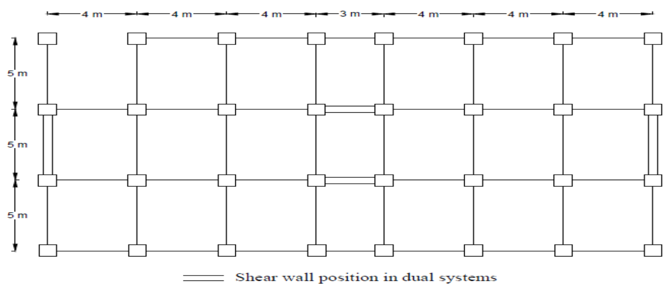

Figure 1. The plan of the modeled buildings

Table 3. Natural period and structural members' properties in modeled buildings

Building Type	T_1	story	column section size (cm)	beam section size (cm)	shear wall thickness (cm)
MRF	0.88	4-6	50x50 (1% reinforcement)	50x50	-
		1-4	50x50 (2.55% reinforcement)	50x50	-
Dual	0.57	3-6	50x50 (1% reinforcement)	40x40	35
		1-2	50x50 (2.55% reinforcement)	40x40	35

In this study OpenSees is used for the non-linear structural analysis. Here two types of material are used. These materials are used to model the behavior of the bars and concrete. Two types of concrete are considered. In OpenSees Concrete02 and Steel02 are used to model the concrete and steel bars. The stress-strain behavior of theses material is shown in Figure 2 and Figure 3.

For considering P-Delta effect in OpenSees, first the gravity loads are applied to structures. The gravity loads are applied statically.

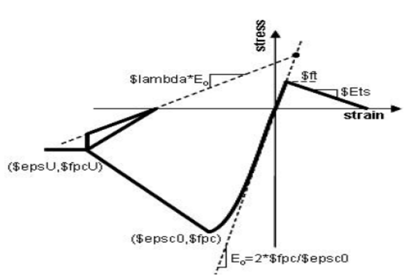

Figure 2. Stress-strain curve of Concrete02 material in Opensees

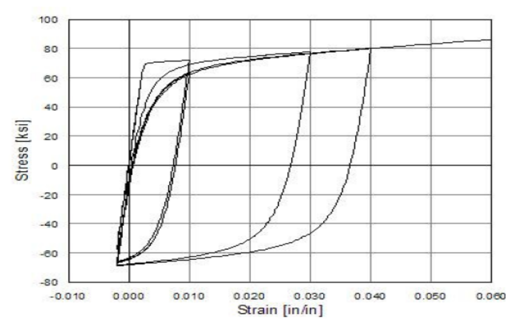

Figure 3. Stress-strain curve of Steel 02 material in Opensees

In this study it is assumed that the frequency content of the input excitations is variable. Three earthquake records are selected. The selected earthquake records are San Fernando (1971), Kobe (1995), and Loma Prieta (1989). The accelerogram of these records are shown in Figure 4 and the Fast Fourier Transform (FFT) is seen in the Figure 5. In the Figure 5, in is seen that the San Fernando earthquake contains a wide range of frequencies from *0.5 Hz* to *5 Hz*. Moreover the Kobe earthquake contains a frequency range from *0.5 Hz* to *3 Hz*. Unlike two other records, the Kobe earthquake does not contain a wide range of frequencies. The dominant frequency content of the Kobe earthquake is concentrated between *1 Hz* and *2 Hz*.

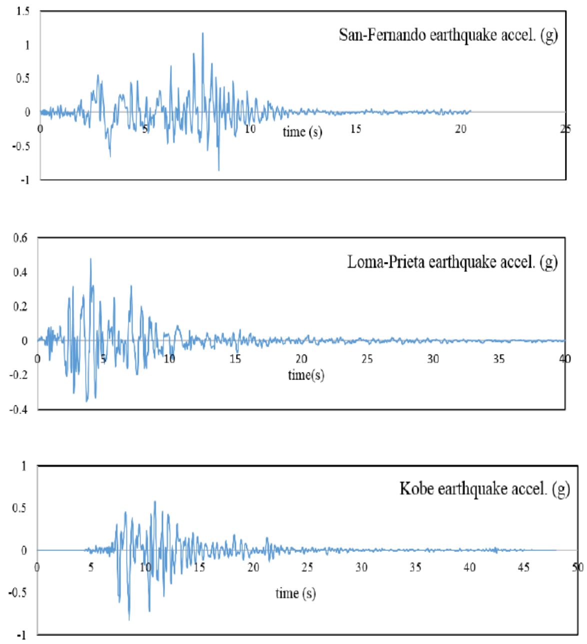

Figure 4. The accelerogram of the selected earthquakes

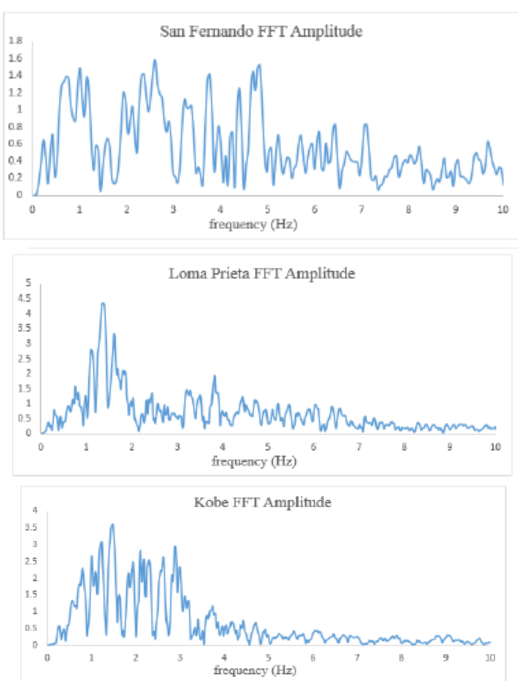

Figure 5. FFT of the selected earthquakes

4. RELIABILITY ANALYSIS AND NUMERICAL RESULTS

Monte Carlo method is one of the most popular methods for reliability analysis in engineering systems. In the Monte Carlo Simulation (MCS) method, the Boolean function is defined as (Dilip, 2013):

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

A pseudo-random number generation algorithm is used to obtain a sequence of N_s input vectors x_i (i=1, …. , N_s). The value of *I(x)* can be 1 or 0. If *I(x)=1*, it means that the failure has occurred in the system. Otherwise the system has not failed.

In the MCS method, the probability of failure can be calculated by (Dilip, 2013):

$$P_f = \frac{1}{N_s} \sum_{i=1}^{N_s} I(x_i) \qquad (2)$$

In this study, for sampling analysis the function $g_i(x)$ must be defined. Two types of function for $g_i(x)$ are considered. In the first type, if the value of function $g_i(x)$ is negative, the failure occurs in the non-structural members. Moreover in the second type, the failure occurs in the structural members.

The $g_i(x)$ functions, are defined according to the maximum inter-story drift. In the first type of $g_i(x)$, the maximum acceptable inter-story drift is *0.005h* and in the second type the maximum acceptable inter-story drift is *0.02h*. The variable *h* is the height of each story. In this study *h* is *3 m* for each story.

The probability of failure for the modeled structures under different earthquake records are shown in the Figure 6 to Figure 10. In the Figure 6, the probability of failure in the moment resisting frame is shown. It is seen that the P_f has a maximum value for each PGA in the Loma Prieta earthquake and the minimum P_f occurs for the San Fernando earthquake. As it was mentioned earlier, the frequency content of San Fernando earthquake has a wide range from *0.5 Hz* to *5 Hz*. The P_f is near to zero for the earthquake with a of *PGA<0.1g*. Also the P_f becomes near to *1.0* for the PGA higher than *0.35g*. The probability of failure can vary if the earthquake excitation is changed. For example, P_f is near to 0.3 in the San Fernando earthquake with a *PGA =0.25g* and P_f is 0.6 for a *PGA=0.25g*. The difference can be up to 30% if the earthquake excitation is changed.

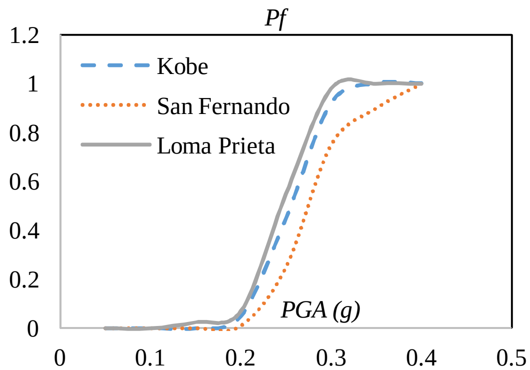

Figure 6. Probability of failure for the moment resisting system.

In the Figure 7, P_f is seen for the dual system. Again it is observed that the maximum P_f occurs for the Loma Prieta earthquake and the minimum P_f occurs for the San Fernando earthquake. In the Figure 8 to Figure 10, the P_f is compared for two structural systems. The P_f is compared for the selected earthquake records. It is seen that the dual system has a better performance under earthquake. The probability of failure can be less than 35% if a dual system is used instead of the moment resisting frame.

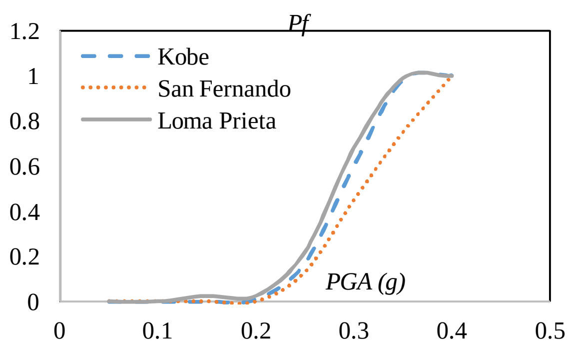

Figure 7. Probability of failure for dual system

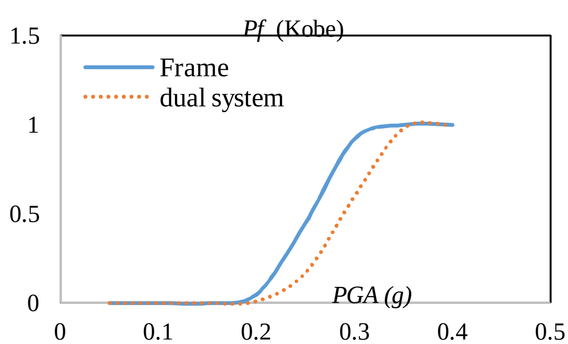

Figure 8. Probability of failure under Kobe earthquake

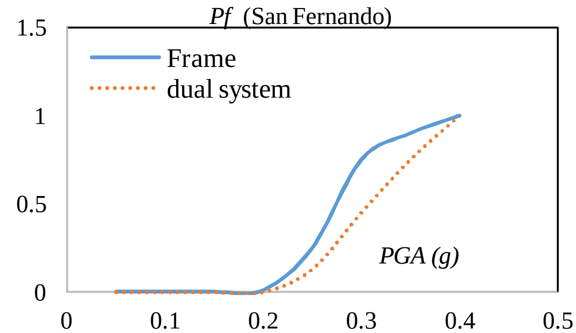

Figure 9. Probability of failure under San Fernando earthquake

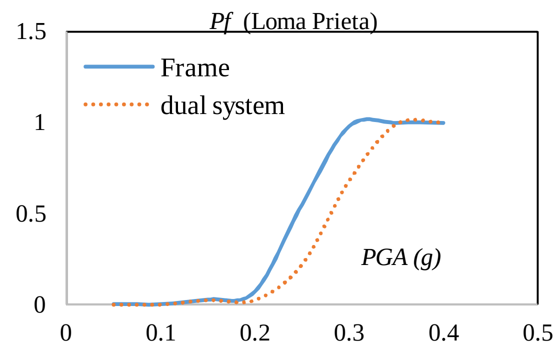

Figure 10. Probability of failure under Loma Prieta earthquake

5. CONCLUSIONS

In this study, the field data were gathered to conduct a reliability analysis of two prevalent structural systems in RC buildings. The field data for the reliability analysis were compressive strength of the concrete and tensile strength of the bars. It was found that the average of the concrete compressive strength in field is less than the assumed design compressive strength in the beams and columns. Also the average of the f'_c is near to assumed design f'_c. Note that the coefficient of variation for f'_c were much larger in the shear walls than beams and columns.

For the bar tensile strength, it was found that the average of the tensile measured strength is more than the nominal tensile strength. For the bars with lower diameter, minimum strength and maximum coefficient of variation were observed. The maximum strength of bars and minimum coefficient of variation were observed for $\phi16$, $\phi18$, $\phi20$ and $\phi22$.

Three earthquake records were selected to conduct nonlinear time history analysis. It was found that the probability of failure can vary up to 30% if the earthquake excitation is changed. Moreover the probability of failure were compared for two structural systems. It was found that the dual system can have better performance and it can decrease the probability of failure up to 35%.

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