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Seismic reliability analysis for strengthening of reinforced-concrete hospital building with base isolation Frames

Mostafa Radmehr[a], Abbas Akbarpour[b], Muhammed Maleknia[c*]

[a]*Lecturer, Civil Engineering, Faculty of Islamic Azad University, (Damavand Branch), Tehran, Iran.*

[b]*Assistant Professor, Civil Engineering, Faculty of Islamic Azad University, (South Tehran Branch), Tehran, Iran.*

[c]*Ph.D Student, Structural Orientation, Faculty of Islamic Azad University, (Chalous Branch), Mazandaran, Chalous, Iran.*

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Abstract

When an earthquake occurs in a region, it causes structural damages to buildings. Retrofitting these damaged structures before earthquakes to happen reduces the potential hazards and minimizes future casualties. In this paper, the seismic assessment of Reinforced Concrete (RC) structures inadequately to withstand seismic loads due to changing demands, and their retrofitting is studied. After a brief introduction of seismic retrofit described for existing structures, methods of assessing the seismic vulnerability of existing buildings are presented. To show the effect of the selected retrofit approach on the seismic behavior of RC buildings, an existing hospital building was selected as the case study. Since the use and occupancy of the selected RC building designed as a residential building had changed, a seismic assessment was carried out using Opensees software to show whether the building needs seismic retrofit or not. After macro modeling to assess the seismic behavior of the structure, the dynamic analysis was conducted by applying 7 earthquake records on soil type B based on Iranian seismic code. Selecting the retrofit approach, which contains using Lead Rubber Bearing (LRB), was performed based on Iranian codes. To consider the uncertainties related to the ground motion, 27 ground motion time histories were utilized and results showed that the building needs seismic retrofit because of the increasing demands. Furthermore, the performed reliability study illustrated an effective role in improving the seismic behavior of the buildings under earthquake loads by the retrofit method.

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Keywords: Reliability analysis, Seismic Assessment, Retrofit, RC Structure, LRB, Monte Carlo Simulation

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* Corresponding author. Tel.: +989113396652; e-mail: mohamad_maleknia@yahoo.com.

Introduction

In the event of an earthquake, structures move in all directions. The most severe damages inflicted upon buildings are caused by lateral movements, which cause the large displacements and disturb the stability of the structure. The importance of seismic assessment and rehabilitation of existing buildings is due to the existence of a large number of inadequate existing structures in earthquake regions. In recent years, quite a lot of research activities have been focusing on this topic and various guidelines and seismic codes have given considerable attention to such buildings [1, 2]. In some cases, the studied buildings by researchers needed strengthening to maintain their performance level for stronger earthquakes, so they are required to be retrofitted. The retrofit method should meet the requirements and criteria by considering the cost, time, and quality. Several approaches such as FRP wrapping of columns and joints, steel or RC jacketing, and shear walls have been developed in previous studies to retrofit RC structures [3, 4]. Many types of research have been undertaking studies to evaluate the performance of the seismic isolation systems which are one of the most practical passive systems in controlling the structural vibrations. Kramer [6] investigated 3-, 7-, and 12-story frames in cases without and with isolator (LRB) using the IDA analysis. Then, the effect of (LRB) isolator was separately determined in the reinforced concrete structures using a fragility curve under 7 earthquakes near and far-fault. Shakib and Fuladgar [7] examined the effect of triple components of earthquake acceleration on the response of the isolated structure in the base by sliding frictional seismic isolator undergoing three components of El-Centro 1940, Tabas 1978, and Northbridge 1994 earthquakes. In the last two decades, quite a lot of researches has concentrated on the field of structural reliability theory. Afshan et al. [8] carried out a reliability analysis on structural stainless steel according to EN 1990 Annex D, they collected a valuable data based on statistical material and structural performance on stainless steel structural elements, finally, they suggested these data for using in reliability analysis of stainless steels. Zhang et al. [9] presented a system-based, analysis method which is introduced as “Direct Design Method” (DDM), in this method a reliability framework has been developed which is a simple First-Order Reliability method versus the costly Monte Carlo method is verified. Hao et al. [10] in this study, is proposed an algorithm of non-probabilistic reliability-based design optimization (NRBDO), The algorithm illustrates that this proposed method applies to structures that are engineering complex as well as the uncertainty distribution of information is not available. Fu et al. [11] used a novel approach to optimize wind resistance in high-rise buildings considering the uncertainties in wind speed, natural frequency of structures, damping and the joint distribution of the wind speed and direction as well as using a new model of earthquake acceleration. The results of this research show that this method could reduce the total weight of the high-rise buildings. Pirizadeh and Shakib [12] provided a framework for improving the performance of special steel moment-resisting frame (SMRF) setback structures based on the reliability method, for this purpose using the results of incremental dynamic analysis (IDA), an algorithm that follows a highly accurate equation, which enables to calculate the maximum inelastic inter-story drift ratio of setback structures. They demonstrated that this economical technique can reduce the potential damage to SMRF frames, which is located in an effective seismic area. Gaxiola-Camacho et al. [13] presented a novel approach for Performance-Based Seismic Design (PBSD) procedure as well as reliability analysis. They applied 20 records on a 9-story steel frame and using 300 deterministic analysis and 600,000 cycles of Monte Carlo simulations (MSC).Their findings, which are criteria the probability of failure for limit states, proved the accuracy of their proposed algorithm. Castaldoa et al. [14] evaluated a concrete structure, which has been equipped by with single-concave friction pendulum, they considered the elastic response pseudo-acceleration corresponding to the isolated period as a one of the variable random and also using Latin Hypercube Sampling for calculation of seismic reliability as well as the seismic robustness. Their results provided appropriate design recommendations for a concrete structure system equipped with FPS.

Risi et al. [15] proposed a novel multi-dimensional limit state function damage criteria for evaluating the seismic performance of reinforced concrete structures. They showed that in a linear state, there is not highly change in fragility and risk curves, while in the nonlinear state for the probability of failure, the range of variation is between 10 to 50 percent. Homami and Aghakouchak [16] studied a novel approach to mixing reliability and fuzzy method to consider the reliability of a rigid steel frame according to the Iranian steel frame codes. They calculated the reliability index as well as presented standard recommendations for using this kind of moment-resisting steel frame, which is widely used in Iranian buildings. Furthermore, many studies have been carried out to assess the reliability of structures with different methods such as fuzzy reliability analysis [17-27], which in this method, the application of fuzzy set theory in structural engineering is investigated. This paper illustrates a study regarding an existing RC building which is a hospital located in Iran. The performance of the structure was evaluated employing linear static and dynamic analyses using OPENSEES 2.6. Structural modeling was based on the Monte Carlo Simulation (MCS) by defining the shear wall as shell elements. Static and dynamic analyses (spectral analysis) have been carried out based on Iranian seismic code (Standard 2800) [5]. Uncertainty is considered in the important structural parameters, including the compressive strength of concrete, the tensile strength of reinforcing bars, and effective stiffness in LRB. For seismic analysis, 27 different models were considered based on the values of uncertain structural parameters from the Monte Carlo Simulation method. These models were analyzed under various earthquake records using OPENSEES. After the seismic evaluation, it was revealed that the building needs to be retrofitted. Since applicability, cost and, time are very important in the rehabilitation methods, two approaches namely steel jacketing and RC shear wall were selected to be used for this structure. Additionally, it was found that both solutions solved the problem partially; because in solution one, some columns were still not strong enough and in solution two the inter-story drift was not allowable. Thus, the final solution is using LRB to reduce the seismic demand of the structure. Besides, to show the effect of structural parameters, a reliability study was undertaken.

2. Structural Modeling

The modeling and analysis of the buildings were conducted via OPENSEES version 2.6, which implements the finite element method for solving partial differential equations [28]. The software has the capability of macro modeling of building structures with high speed in the nonlinear dynamic analysis. OPENSEES 2.6—the nonlinear analysis program—was used to model the CM building to calculate the vulnerability. At first, the existing building was modeled by SAP2000. Then, to show the damage to the building in the nonlinear dynamic analysis under the earthquake records, it was modeled in OPENSEES 2.6. Structural information in this research as a case study model has been considered as the total area of the building is 9600 square meters and also the rectangular shapes were used for the beams (40x40, 40x45 and 40x50 square centimeters) and columns (40x40, 40x45, 40x50, 40x55, 40x60, 40x65, 40x70 and 40x80 square centimeters), and reinforcement bars for beams was φ20 and for columns were φ20 and φ22, with different reinforcement bar numbers. Furthermore; the shear walls were modeled with shell elements to consider out-of-plane stiffness. Fig.1 shown the configuration of column and shear walls in this research. Since the building has already been constructed, it is necessary to examine the material properties, especially the concrete compressive strength; and that is due to the low quality of cast-in-situ RC buildings. Therefore, one concrete core test and two Schmidt hammer tests in each story have been performed randomly. The results show that the lowest, highest and average value of concrete compressive strength are 14, 21, and 18 N/mm2 respectively, hence the average value—18N/mm2—was used in the structural modeling and assessment. The yield stress for longitudinal reinforcements and stirrups (fy) were assumed 400 and 300 N/mm2, respectively also the unit weight of concrete and the Poisson's ratio for concrete considered 24 kN/m3 and 0.15 respectively. To protect the reinforcement against corrosion and fire concrete cover to reinforcement bars based on Iranian concrete code it was applied 50 mm.

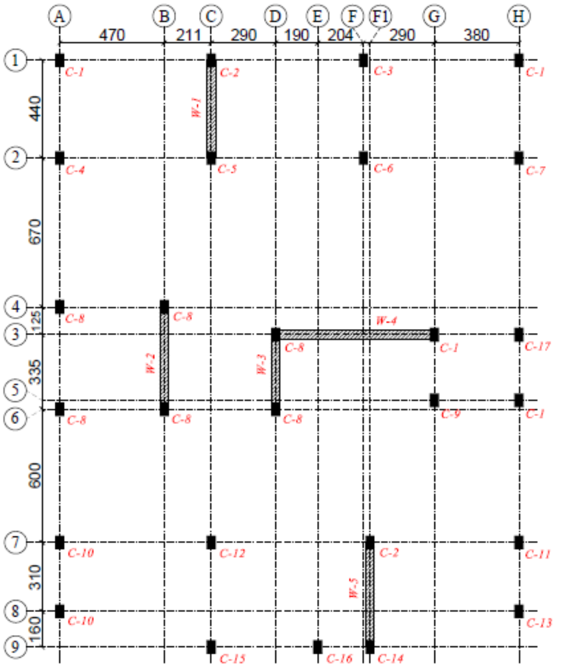

Fig. 1. The plan of the existing building and labeling of columns and shear walls

3. Sampling Methods

Various techniques are available to calculate the key parameters in the structural reliability method, in which sampling methods are one of the most important ones. The main advantage of the sampling methods is evident when the closed-form of the limit-state function is not available for the structure. Enhancing the safety and limiting the uncertainties in the design of structural systems is a matter of importance. The structural reliability is employed as a probabilistic criterion to evaluate the reliability of structural systems. To describe the system behavior and identify the relationship between the basic parameters in a structural system, the limit state function of the system has to be determined. Using the time-invariant reliability analysis procedure, the probability of failure can be determined by Eq. 3.1. In which, $g(X)$ is the limit state function, Pf the probability of structural failure, and $f_x(X)$ is defined as the probability density function.

$$Pf = P[g(X) \leq 0] = \int \ldots \int_{g(X)\leq 0} f_x(X)dX \tag{3.1}$$

This equation can be rewritten as below:

$$Pf = \int \ldots \int I\,[X]\, f_x(X)dX \tag{3.2}$$

Where, $I[X]$ is an indicator function $I(x_i)$ is defined in Eq. 3.3, which is regarded as an indicator for considering successful or unsuccessful simulations.

$$I\,[X] = \begin{cases} 1 & if \quad g(X) < 0 \\ 0 & if \quad g(X) \geq 0 \end{cases} \tag{3.3}$$

Evaluating analytically for complex or large-scale structures is practically impossible since there are no closed-form expressions, the best technique is the numerical solution method by the Monte Carlo Simulation (MCS). Although MCS has a high computational cost, it is regarded as an effective method and is widely utilized for the evaluation of the probability of failure in computational mechanics, either to compare with other methods or as a self-contained reliability analysis tool. When the analytical solution is not feasible and the limit state function cannot be expressed or estimated analytically, the MCS technique is used. This method is mostly the case in complex problems with a large number of basic variables where other reliability analysis methods are not suitable. If we express the limit state function as g(x) < 0, which is the vector of the random variables. The joint probability of failure for all random variables is defined by Eq. 3.4, which gives an unbiased estimator of the probability of failure.

$$Pf = \frac{1}{n}\sum_{i=1}^{n} I\,[x_i] \tag{3.4}$$

In structural reliability, while using simulation methods, evaluating the probability of failure is important for a specified performance function efficiently and accurately. With the help of limit state function G = C-D, which in this study defined by subtraction of the maximum inter-story drift as the demand (D) from the allowable inter-story drift (C) which is limited to the life safety limit according to FEMA 356 as the capacity, it is easy to determine the probability of failure from relation 3.5. In this respect, is the number of times that the function G is negative (failure) to N which is the total number of iterations of analysis.

$$Pf \cong \frac{N_s}{N} \tag{3.5}$$

Finally, Reliability Index is determined by Eq. 3.6.

$$\beta = \varphi^{-1}(Pf) \tag{3.6}$$

Although the mathematical formulation of MCS is relatively simple and can handle practically every possible case, regardless of its complexity, the computational effort involved in conventional MCS is excessive. For this reason, a lot of sampling techniques, also called variance reduction techniques, have been developed to improve the computational efficiency of the method by minimizing the sample size and reducing the statistical error that is inherent in MCS. Among them is the importance of sampling Latin Hypercube Sampling provides a limited sampling plan instead of random sampling in the direct simulation. In this method, the interval from 0 to 1 is divided into N equal parts and one sampling point is generated from each part, which the probability of events is the same in each part. To generate input random variables inverse transformation is used which have been shown by Eq. 3.7.

$$u_i = \frac{u}{N} + \frac{i-1}{N} \tag{3.7}$$

In which, u is a random parameter in the interval (0, 1) and $u_i (i = 1,2, \ldots, N)$ is the random value for the i-th interval. Then, a value of each input variable is selected accidentally and put into the limit state function. This procedure is repeated for N times to calculate of probability of failure.

4. Static Analysis

The model was analyzed for dead, live, and seismic loads using Opensees. The assessment and retrofit designs have been carried out according to the requirements of Iranian codes. As the building was used to be residential, now that it is used as a hospital building the live and dead loads would be greater. Dead load is 550 kg/m2 and live loads are 300, 350, 400 and 500 kg/m2 based on Iranian codes. The seismic load is defined as shown in Fig. 2 according to the equivalent static load. Because the building use has been changed from residential to the hospital and the period of the structure has been changed due to strengthening, hence two main differences in the building before and after retrofitting are: Importance factor (I) and Spectral parameter (B). To consider dynamic analysis, the spectrum should be defined based on Standard 2800 and soil type II (375<Vs<750). The spectrum is shown in Fig. 2.

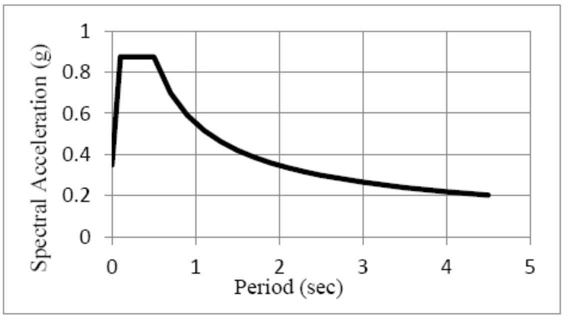

Fig. 2. The spectrum curve for dynamic analysis

Based on Standard 2800, the maximum inter-story drift should be less than 0.45%. However, Story 4-15 had higher drift than the allowable value in X and Y directions. Table 1 shows the drift ratio in X and Y directions. To check the torsion of the building, eccentricity in X and Y directions should be less than 20% of the width and length of the building in the plan. By considering 21.07m wide and 26.7m long, the allowable eccentricities are 4.2 m and 5.34 m in X and Y directions respectively. The results show that the eccentricity in the X direction is exceeding the allowable value. There is no problem regarding the overturning and stability of the structure. Besides, the allowable DCR is one, whereas in many columns and shear walls DCR is higher than one. It should be noted that vertical seismic loads, the simultaneous effect of seismic loads in both directions, and allowable beams deflections, have not been considered in the existing structure design.

5. Dynamic Analysis

Since DCR in some elements is greater than the allowable value prescribed by the seismic code, the dynamic analysis must be carried out to evaluate the structure and see if it requires retrofitting. In this paper, time history analysis is employed to perform the nonlinear dynamic analysis based on 7 earthquake records on soil type B, which is the soil type on which the building is built. Table 1 shows the selected records which are taken from FEMA-P695 [29]. Having been selected based on the soil type and frequency content, records were scaled to a standard design spectrum using Iranian seismic code (Fig. 3).

Table 1. Selected earthquake records on soil type B

Name, Station	Vs (m/s)	USGS soil type	Fault	R (km)	M	PGA (g)
Hector Mine, Hector SCSN	685.00	B	Strike-slip	26.50	7.10	0.27
Kobe, Japan Nishi-Akashi CUE	609.00	B	Strike-slip	8.70	6.90	0.51
Kocaeli, Turkey Arcelik KOERI	523.00	B	Strike-slip	53.70	7.50	0.22
Manjil, Iran Abbar BHRC	724.00	B	Strike-slip	40.40	7.40	0.13
Chi-Chi, Taiwan TCU045 CWB	705.00	B	Thrust	77.50	7.60	0.51
Friuli, Italy Tolmezzo	425.00	B	Thrust	20.20	6.50	0.35
Tabas, Dayhook	659.00	B	Thrust	13.90	7.35	0.33

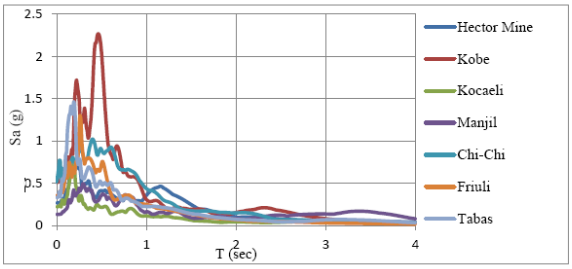

Fig. 3. Earthquake spectra on soil type B

Maximum inter-story drift, as one of the most important structural parameters indicating the performance of structures, was chosen to represent the seismic performance of the building. After applying 7 earthquake records, therefore, the maximum inter-story drift of the structure was calculated and compared before and after the retrofit. To show the performance level of the structure, 3 levels were considered based on FEMA 356 [29]. According to this standard, attributed drift to the level of Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) were 1%, 2% and 4% for RC frames respectively. The allowable seismic performance level of hospitals is IO and all of the non-structural elements should not have a performance level higher than this level. In other words, the maximum inter-story drift of this building must be lower than 1%. Fig. 4 and Fig. 5 shows the response of the structure before retrofitting in each direction.

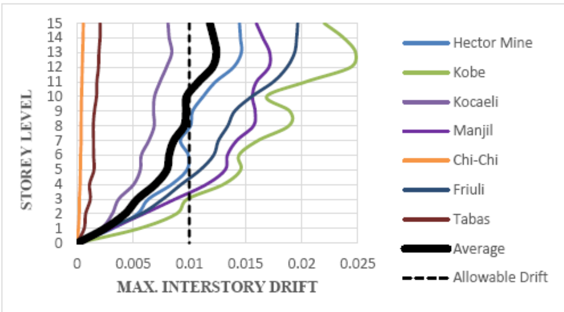

Fig. 4. Maximum inter story drift before retrofit in X direction

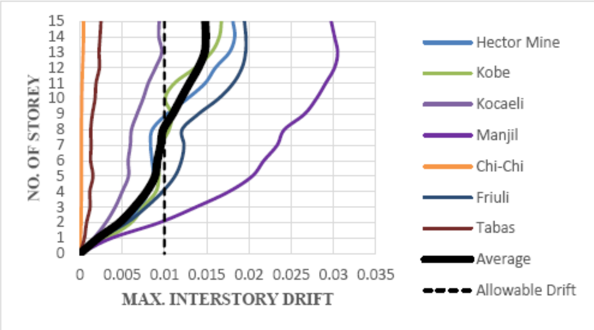

Fig. 5. Maximum inter story drift before retrofit in Y direction

The maximum inter-story drift diagrams show that the performance level of the structure is not IO which is not allowable for hospitals. Performance level in the lower stories story one to 8 is acceptable (IO). However, in the upper stories, it is not acceptable because the performance level is LS. Fig. 4 shows that Kobe earthquake causes the most damages to the structure in the X direction, whereas Manjil is the most vulnerable ones in the Y direction.

6. Retrofit Plan

To retrofit RC structures, there are several methods such as FRP wrapping of columns and joints, steel or RC jacketing, shear walls, and so forth. In each retrofit procedure, one of them has to be selected by considering all factors such as efficiency, cost, time of execution, and so on. Among all the methods, two approaches, steel jacketing and shear wall, were selected due to their low cost as well as execution time and high efficiency in RC building, especially in Iran. To show the retrofit plan of this structure, some steps are considered to yield the best results, at first, decreasing the total horizontal seismic load, secondly, reducing drifts and eccentricity of building as well as reducing DCR of all elements, in the following improving all defects in execution and finally rehabilitation of the foundation have been considered. In the first step, the best solution to decrease the total horizontal seismic load is adding shear walls on top of the foundation to increase the base shear and decrease the seismic load. These shear walls also can be known as retaining walls to withstand soil pressure. By adding retaining walls in the 1st and 2nd stories, unallowable drifts and eccentricity of the building were solved. In addition, since these shear walls carried the shear loads in these levels, DCR of all the elements decreased.

For the rest of the stories, there were two main problems—unallowable drifts and high DCR in columns. Each selected retrofit approach, shear walls and steel jacketing, does not solve all the problems solely. In other words, although steel jacketing can reduce DCR in columns, it cannot reduce drift perfectly. On the other hand, shear walls can reduce drift while some columns fail under seismic loads. Thus, the best result is a combination of two selected retrofit approaches.

First of all, some shear walls were added in all stories to reduce drift, eccentricity, and DCR in columns in which architectural limitations such as creating openings in shear walls as facades were also considered. After designing the added shear walls and reducing the drift and eccentricity solely to the allowable values, some columns still had high DCRs. Steel jacketing solves this problem by confining concrete, making a composite section. Fig. 6 and Fig.7 show the plan of the building retrofitted by shear walls and steel jacketing.

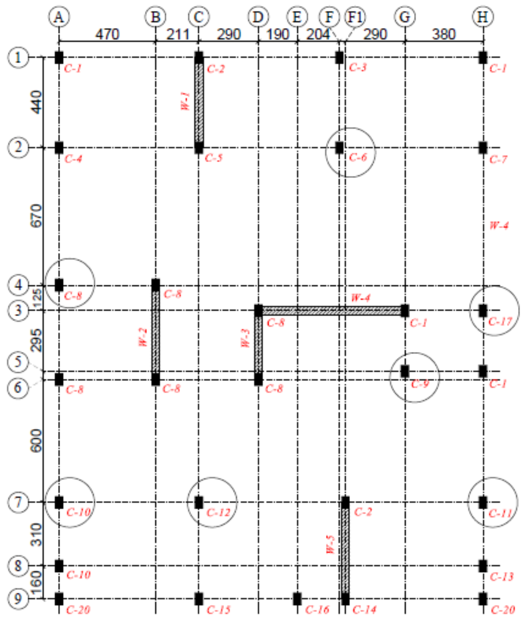

Fig. 6. Plan of the building retrofitted by shear walls and steel jacketing

Fig. 7. The building retrofitted by steel jacketing

As shown in Fig. 6, column C20 was added to the structure to complete the surrounding shear walls in stories 1 and 2. One of the main issues in adding shear walls to an existing RC building is the connection to the foundation. The next step in the retrofit plan is remedying all the defects caused by the faulty execution of the building and construction work. As mentioned before, the test results of the compressive strength of concrete show that the average value is 18 MPa, which was used in the modeling. After retrofitting the structure, the final step is the rehabilitation of the foundation because adding and increasing new loads may cause higher demands in the foundation. To solve these problems, it should be mentioned that using LRB can be useful in reducing demands instead of increasing the capacity of strengthening of structural elements.

7. Base Isolation

The use of isolators with a performance similar to that of horizontal springs decreases the earthquake forces and resonates with dominant frequency content by changing the inherent period of structures. The concept of increasing the period of a structure is the same as using isolators [30]. Seismic isolation is one of the many different ways to resist the earthquake load, which increases the time period of structure and force transfer interruption path [31]. The lead-rubber isolator has been more prevalent among isolator systems and has been widely used in many countries. These isolators contain some layers of steel and rubber plates with the lead bar which is embedded in a few holes. The lead cores deform at shearing stress with a magnitude of 10 MPA, expressing a hysteresis behavior with a two lines response generated in the lead (Fig. 8). The role of this lead core is energy dissipation, which ultimately decreases the amount of isolator displacement; thus, it can be called an auxiliary damper [32] (Fig. 8).

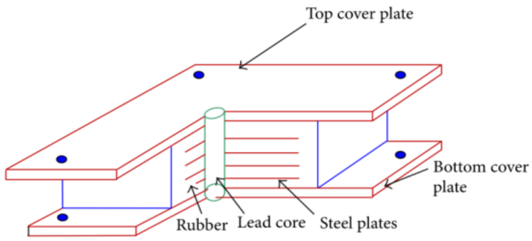

Fig. 8. Schematic of Rubber isolators with lead core [32].

In this study, the used isolator system is the rubber isolator with lead core. To design the isolators, the effective damping, design time period, effective horizontal stiffness, and the maximum horizontal displacement of support have been calculated based on design shear strain. Isolators have been designed based on FEMA356 for these weak hospital buildings. The LRB hysteretic model was assumed to be bilinear with first stiffness k1 and secondary stiffness k2 as illustrated in Fig. 9. The ratio of the primary stiffness to the secondary stiffness (k1/k2) is assumed to be 10 [33, 34]. The characteristics of designed LRB isolators are listed in the next section based on Monte Carlo simulation.

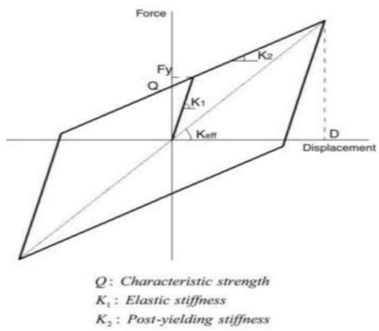

Fig. 9. LRB hysteresis behavior

The analytical spring models (zero length element) which is usually employed for simulating the behavior of LRB devices, were constructed with the OpenSees (Fig. 10). In this study, “KikuchiAikenLRB” material which produces nonlinear hysteretic curves of lead-rubber bearings has been assigned to the spring.

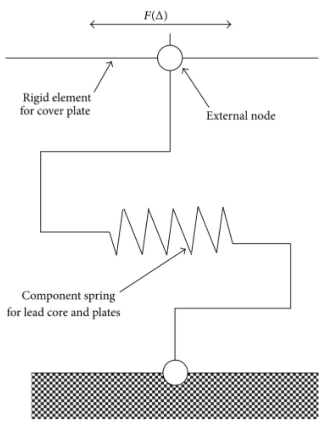

Fig. 10. Analytical model for component spring (LRB model) [35].

8. Results And Descussion

To consider uncertainty in structural parameters, first, a probabilistic distribution was assigned to each of them [36]. Then, by using the Monte Carlo Simulation method [36], 3 different values have been generated for each structural parameter. These parameters, with their assigned average value, type of distribution function, and coefficient of variation, are shown in Table 2. The value of average compressive strength of concrete was assumed based on Iranian seismic code (Standard 2800) and Iranian concrete code [5]. The base isolation design parameters have presented in Table 3.

Table 2. Random variables

Parameters	Average Value	Standard Dev.	Distribution Function Type
Compressive Strength of Concrete	25 MPa	3	Normal
Tensile Strength of Reinforcing Bars	400 MPa	20	Log-Normal
Effective Stiffness of LRB	3000 kN/m	1000	Normal

Table 3. Design of base isolation

ξ	Td (sec)	Q/W	K eff (kN/m m)	DD (m)	Ap (cm2)	A0 (cm2)
0.1	2.5	0.03	5679.1296	0.30535	331.0875	11803.48
0.26	2.5	0.06	5679.1296	0.229013	662.175	11803.48
0.48	2.5	0.09	5679.1296	0.185061	993.2625	11803.48
0.19	4	0.03	2218.41	0.398824	331.0875	11803.48
0.52	4	0.06	2218.41	0.293136	662.175	11803.48
0.78	4	0.09	2218.41	0.293136	993.2625	11803.48
0.3	5.5	0.03	1173.373884	0.474191	331.0875	11803.48
0.71	5.5	0.06	1173.373884	0.403062	662.175	11803.48
1	5.5	0.09	1173.373884	0.403062	993.2625	11803.48

After seismic evaluation and determining the unallowable values of structural parameters such as DCR and Drift ratio, seismic rehabilitation was used to reduce the demands and meet the criteria of the regulations. By using LRB, the result of this procedure can be shown as follows based on the mentioned steps:

1. As mentioned before the low compressive strength of concrete was considered in modeling.

2. 27 models were evaluated with 3 different structural parameters based on Monte Carlo method.

3. By using steel jacketing in retrofitting columns, all DCRs were reduced below 1. Some columns had allowable stresses due to adding shear walls and the 12 remaining columns were strengthened by using steel jacketing.

4. Due to the soil loads, a retaining wall surrounded the building so that the base level was changed to be at the second story.

5. Retrofit approach causes the seismic performance of the structure to be IO level in the hospital based on FEMA-356. Fig. 11 and Fig. 12 show the response of structure after retrofitting in each direction.

After seismic assessing, designing, and retrofitting of the hospital building, the instructions were given to the retrofit constructor to execute all of them.

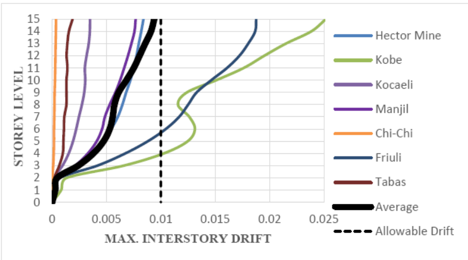

Fig. 11. Maximum inter story drift after retrofit in X direction

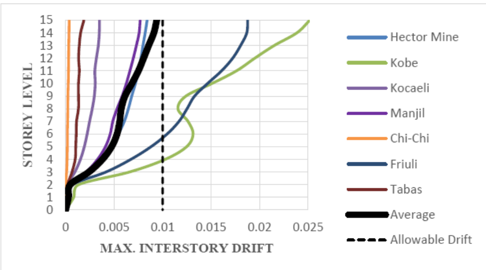

Fig. 12. Maximum inter story drift after retrofit in Y direction

As illustrated in figures above, time history analysis after retrofit shows that average drift of structure is less than 1% in both directions. Because of changing the first mode behavior of the structure and its period, only in two earthquake records (Friuli and Kobe) the maximum drift is higher than allowable value. Moreover, the drift of story 1 and 2 is about zero, which is because of the retaining wall that reduces the drift considerably in these two stories. The maximum inter-story drift diagrams show that the performance level of the structure is not IO which is not allowable for hospitals. Performance level in the lower stories story one to 8 is acceptable (IO).Finally, to show the effect of random analysis of structural parameters and in order to calculate the reliability index of systems, two different steps can be presented, the performance failure functions as (G=Capacity - Demand) which demand was considered as the maximum inter-story drift and Capacity was regarded as the allowable inter-story drift according to Iranian seismic code (Standard 2800), also in order to consider capacity and demand, Monte-Carlo sampling technique was employed. However, it must be considered that extracting this close form failure function will not be easy and most of the time is available only for linear elastic system. Also by using dynamic nonlinear analysis, the seismic demands and structural resistance are evaluated under earthquake excitation. It would be practical to conduct nonlinear time-history and then calculate desired performance level or failure. By considering average values of all 27 random models under 7 earthquake and drawing the probability density function (PDF) for the performance failure functions (G) shown in Fig. 13 and Fig. 14 the reliability factor has been calculated by Eq. 3.6 which are 1.48 before retrofitting and 1.96 after retrofitting in X direction, and 1.28 and 1.59 in Y direction, which also presented in Table 4 and Table 5.

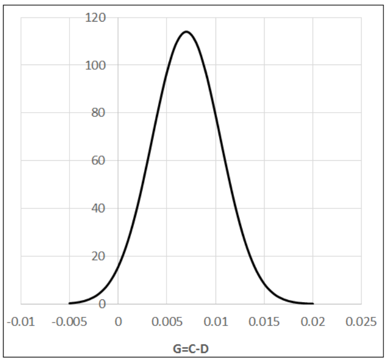

Fig. 13- Performance failure functions (G) in X direction

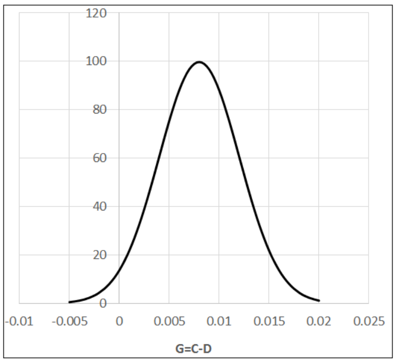

Fig. 14- Performance failure functions (G) in Y direction

Table 4. Performance failure functions (G) in X direction

Statistical characteristic	**G (Limit-state function)=C-D**
distribution function	**Normal**
Mean	0.0068
Standard deviation	0.00346
Reliability index in X direction	1.96

Table 5. Performance failure functions (G) in Y direction

Statistical characteristic	**G (Limit-state function)=C-D**
distribution function	**Normal**
Mean	0.0075
Standard deviation	0.0047
Reliability index Y direction	1.59

9. Conclusion

A seismic evaluation was performed to assess the vulnerability of a 15-story hospital building, which had been a typical residential apartment and changed to be used as a hospital. This assessment was undertaken using Opensees software based on macro modeling. The low compressive strength of concrete in tests Schmidt hammer, and Core test, and static as well as dynamic analyses show that the building must be retrofitted. Because applicability, cost, and time are very important in rehabilitation methods, using LRB is selected as the retrofitting approach. In addition, by considering 5 steps as the strategy of the retrofit procedure, all problems such as high base shear, unallowable drift, eccentricity, DCR, and defects in construction have been solved. To show the seismic performance of the structure, time history analysis was carried out using 7 earthquake records on soil type B. The maximum inter-story drifts before retrofit is more than 1% and not allowable for hospitals because this drift is within LS performance level. However, after retrofit, this value is reduced to 1% so that the seismic performance level of the structure is IO. It should be mentioned that the maximum drift in the lower stories surrounded by retaining walls must be about zero so that the base level would be on story 2. Furthermore, retrofitting and strengthening of structures change structural parameters so that they might be damaging for structures. For instance, before retrofit, Manjil and Kobe earthquake records inflict the most damages on the structure. However, after retrofit, Kobe and Friuli are the most vulnerable ones.

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