Magazine of Civil Engineering

ISSN 2712-8172

journal homepage: http://engstroy.spbstu.ru/

Research article

UDC 624.046

DOI: 10.34910/MCE.117.1

Effect of uncertainties of shear wall on reliability of rehabilitated structure

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Keywords: special steel moment frame, steel shear wall, uncertainty, seismic rehabilitation, probabilistic seismic performance, fragility curve

Abstract. Uncertainties in seismic demand and structural capacity create conservatism in acceptance criteria for structural performance levels. In case of such conservations, rehabilitation costs can increase, and seismic evaluation results of the structures may be incorrect. In design and rehabilitation of structures, considering uncertainties and reducing them can decrease existing conservations and result in optimal design. Seismic rehabilitation guidelines use reliability index coefficients to consider the uncertainties of the existing structure. In seismic rehabilitation of structures, the existence of uncertainties in the secondary system, which are added to seismic rehabilitation of the existing structure, can lead to an increase in the existing uncertainties. Therefore, in this paper, on the example of rehabilitation of a 3-story steel frame structure of the SAC project by a steel shear wall, quantifying of uncertainties of steel shear walls was considered, and a parametric study of the reliability index of the structure was done. The studied structure was modeled in OpenSees software and was analyzed in the presence of uncertainties of the steel shear wall before and after rehabilitation. Based on the performed analysis and considered uncertainties, values of the reliability index of the rehabilitated structure by the steel shear wall were calculated. According to results, rehabilitation of structure reduced the maximum inter-story drift ratio and the probability of failure, while consideration of uncertainties of rehabilitated structure increased the maximum inter-story drift ratio and the probability of failure, and therefore existing conservatism dropped.

Citation: Maddahi, M., Gerami, M., Naderpour, H. Effect of uncertainties of shear wall on reliability of rehabilitated structure. Magazine of Civil Engineering. 2023. 117(1). Article no. 11701. DOI: 10.34910/MCE.117.1

1. Introduction

In recent decades, the performance-based seismic evaluation method has been continuously updated. ATC [1] began work on next-generation performance-based seismic design criteria under the ATC-58 project for new and existing structures. In addition, the FEMA 445 [2] released a report under the title "Guidelines for Next-Generation Methods". The main purpose of the next-generation methods was to examine conservatism of acceptance criteria and to modify the structural performance evaluation. In developing seismic evaluation methods based on performance, the existence of conservatism in the acceptance criteria of structural performance levels has increased the cost of retrofitting and reconstruction. Also, the results of the evaluation of the seismic performance of the structures are incorrect due to the existing conservatism. Hence, researchers analyzed the seismic evaluation of designed structures according to the old regulations in an analytical and laboratory manner. The results of the studies indicated a significant conservatism in the acceptance criteria of the old regulations. Therefore, this issue expressed the need for considering uncertainties in the estimation of structural engineering issues [3–6].

When the structural response is simulated, active loading on the system and resistance of the member cannot be definitively expressed in the presence of uncertainties. However, the possible range of demand and resistance can be predicted and idealized. Consequently, a probabilistic design philosophy can provide a level of safety for engineered structures with the prediction of the failure probability. Therefore, definitive methods of structural design have been replaced by the reliability and risk-based methods, which have been spread around the world. Probabilistic methods of analysis of structures considered system properties and external forces as uncertain random variables [7]. The uncertainties in the probabilistic methods were categorized as aleatory and epistemic [8]. Aleatory uncertainty is also called random uncertainty or Type-A uncertainty, which is due to random changes of phenomena. As a result, these types of uncertainties cannot be reduced. In contrast, epistemic uncertainty is also called reducibility uncertainty or Type-B uncertainty, which is due to the limitation of knowledge or data. These types of uncertainties can be reduced by improving the mathematical models or increasing data collection.

Zareian and Krawinkler [9] proposed a probabilistic approach to minimize the failure potential of the structural systems based on various sources of uncertainty. Helmerich [10] investigated the reliability of steel structures and showed that structural resistance can be predicted by using appropriate models of material properties and geometric parameters. These studies have shown that a better description of the model uncertainties is necessary. Currently, the used probabilistic models for model uncertainties are largely based on visual judgment and limited information. Shafei et al. [11] presented a simple method for predicting the mean and dispersion of variables in the moment frame systems subject to seismic stimulation. Based on studies, the researchers considered probabilistic models for uncertainties and analyzed the structures. Torabian and Taghikhany [12] used the single-degree of freedom system to investigate the uncertainties of steel moment frames. They investigated the aleatory and epistemic uncertainties of structures using incremental dynamic analysis and the Latin Hypercube Sampling method.

Holicky et al. [13] tried to improve the model uncertainty definitions and proposed a general methodology to determine their quality by comparing the model and empirical results. They studied the implications of the model uncertainty for existing and new structures and provided examples based on recent studies on the flexural and shear strength of reinforced concrete beams. Asgarian and Ordoubadi [14] considered the parameters of the model and examined the effect of uncertainty of equivalent damping, effective seismic mass, yield strength, and ultimate strength of the steel structure. Hajirasouliha et al. [15] studied the effect of uncertainty in structural properties and ground motion on the performance of braced frames. In addition, Zhang et al. [16] studied uncertainties in steel structures such as uncertainty in cross-section and structural loading.

Dyanati et al. [17] considered the demand and capacity uncertainties of the braced frame system and examined the seismic performance of structures. Jiang and Ye [18–19] considered different sources of aleatory and epistemic uncertainties for seismic risk assessment of the structures. They modeled random variables by using the Latin Hypercube Sampling method. They showed that the uncertainties have a significant impact on seismic risk assessment of the structures, which cannot be neglected. Piluso et al. [20] developed the seismic design method of moment resisting frames with considering the uncertainties of the material properties and the theory based on the failure mechanism. Norouzi and Gerami [21] studied the effect of uncertainty in characteristics of the ground motion on the performance of steel moment frames. Astroza et al. [22] proposed a new approach to reduce the modeling uncertainty. They considered three-story steel frame structure with modeling uncertainty such as geometry, inertia properties, gravity loads, and damping properties. Wijaya et al. [23] considered the steel structures with the hysteretic dampers, and investigated uncertainties using the Latin hypercube Sampling method.

These researches were done in the seismic design of structures considering uncertainties in the designed structures. In seismic regulations such as FEMA P695 [24], the aleatory and epistemic uncertainties are considered separately. In these regulations, uncertainties are considered based on qualitative data in existing structures. In seismic rehabilitation of structures, the secondary system, which is added to the existing structure also has uncertainties. These uncertainties may increase the uncertainties of the rehabilitated structure.

In this study, a steel shear wall was used for rehabilitation of the moment frame. Then, uncertainties of the steel shear wall and quantification of them were investigated to evaluate the uncertainties in seismic rehabilitation. The design and evaluation of the seismic performance of structures should be performed using reliability-based methods. In this study, the effect of uncertainties on the seismic performance of the rehabilitated steel moment frame was studied using the reliability methods.

2. Methods

2.1. Specifications of the selected structural model

In this paper, a 3-story steel moment frame was selected [25]. This structure, designed in accordance with the UBC94 regulation, met the requirements of gravity and seismic design. The structure was designed as a special moment frame and an office building located on stiff soil in Los Angeles. In this structure, the perimeter moment frames were considered as a structural system. The plan and height of the desired structure are indicated in Fig. 1. As shown in Fig. 1, the structure is three-dimensional. Because three-dimensional modeling of the structure increases the volume of the problem, the structure was modeled two-dimensionally. In a two-dimensional model, an equivalent gravity column was used to reduce the uncertainties caused by simplification.

There were rigid connections in the lowest floor of this structure. The damping of the structure was 2 %, and the yield strength and modulus of elasticity of the beams and columns were 50 kip and 29000 ksi, respectively. The sections assigned to members of the structure are presented in Table 1. The regulations, published after the Northridge earthquake, proposed design recommendations for the creation of a plastic hinge in the beam and away from the column surface. Here, the displacement of the plastic hinge away from the surface of the column was done by increasing the beam capacity in the column surface via adding the cover plates on the flange of the beam. The bilinear moment-rotation relationship with strain hardening of 3 % was used to define the plastic behavior of beams and columns. Rigid elements with rotational spring were used to model the panel zone in the OpenSees software [26]. In this model, the trilinear shear force-shear strain relationship was used to describe the shear behavior of the panel zone. Details of the panel zone are indicated in Fig. 2.

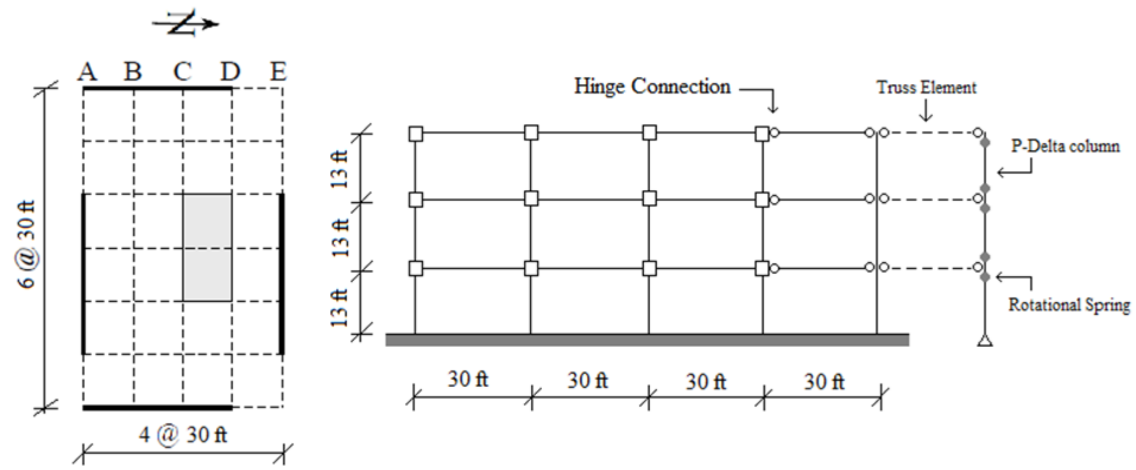

Figure 1. Plan and height of the 3-story structure [25].

Table 1. Sections assigned for the studied structure.

Story	Moment Resisting Frame (MRF)			Gravity Frame (GF)	
	Columns		Beam	Column	Beam
	Exterior	Interior			
1	W14×257	W14×311	W30×116	W14×68	W16×26
2	W14×257	W14×311	W30×116	W14×68	W16×26
3	W14×257	W14×311	W24×62	W14×68	W14×22

2.2. Rehabilitation of the selected structure

Based on the performed evaluations on the structure, its weaknesses were determined. The use of appropriate methods is essential for correcting the weaknesses and performing the rehabilitation. For rehabilitation, according to requirements of the structure, the proper strategy should be selected. In this paper, the steel shear wall was considered for rehabilitation of the existing structure. Different methods are available to design and analyse steel shear walls, and the strip model method is one of the most popular ones. This method includes a set of stretching strips and it was first suggested by Thorburn et al. [27]. Fig. 3 shows an example of stretching strips that is equivalent to a steel shear wall in the strip model.

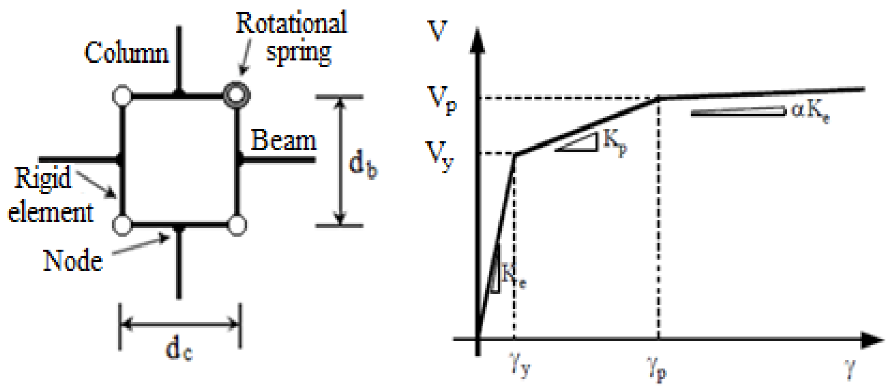

Figure 2. Analytical model and the trilinear shear force–shear strain relationship for panel zone [25].

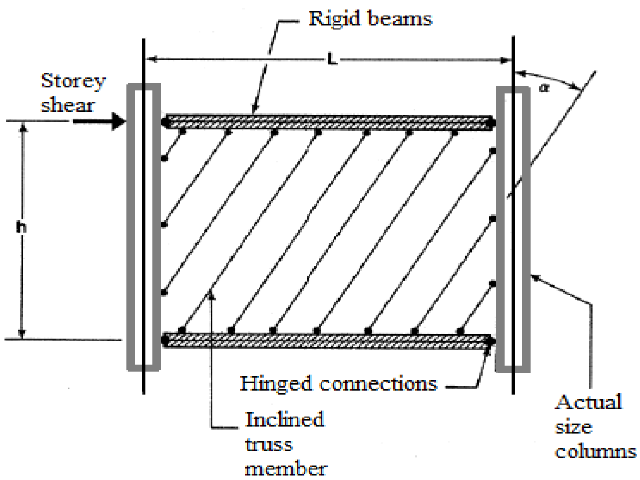

Figure 3. An example of the strip model of the steel shear wall [27].

In the strip model method, the shear wall is equivalent to a brace. Then the thickness of the steel shear wall is calculated according to Eq. (1):

$$t_w = \frac{2A_g \Omega \sin\theta}{L \sin 2\alpha}. \tag{1}$$

Here, the parameters of θ, A_g and α are the angle between the equivalent brace and the vertical line, the cross-section of the equivalent brace, and the tensile field angle, respectively; Ω is the over strength coefficient of the steel shear wall that is equal to 1.2.

Timler and Kulak [28] presented Eq. (2) for the estimation of the tensile field angle (α), which includes geometric properties of the steel shear walls and boundary elements.

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w L}{2A_c}}{1 + t_w h\left(\dfrac{1}{A_b} + \dfrac{h^3}{360 I_c L}\right)}. \tag{2}$$

Here, parameters of L, h and t_w are the width of the frame span, the height of the floor, the thickness of the steel shear wall, respectively; A_b and A_c are the cross-sections of the beam and the column; I_c is the moment of inertia of the column. This equation is used to design steel shear walls by American and Canadian regulations [29–30]. After determining the thickness of the steel shear wall and by considering the appropriate number of tensile strips, the cross-section of these strips in each floor is calculated using Eq. (3):

$$A_s = \frac{L\cos\alpha + h\sin\alpha}{n} t_w. \tag{3}$$

In this equation, the parameter of n is the number of strips per span. To describe the behavior of steel shear walls, at least ten tensile strips were placed in each span. After the design of the shear walls Eqs. (4)–(5) must be controlled to prevent the buckling of the columns along the shear wall in each floor:

$$I_c \geq \frac{0.00307th^4}{L} \tag{4}$$

$$M_{fpc} \geq \frac{\sigma_{ty} th^2}{4}\cos^2\alpha, \tag{5}$$

where M_{fpc} and σ_{ty} are the plastic moment of the column and the tensile field stress, respectively. In addition, Eq. (6) must be controlled to prevent bending of the beam in each floor:

$$M_{fpb} \geq \frac{\sigma_{ty} tL^2}{8}\sin^2\alpha, \tag{6}$$

where M_{fpb} is the plastic moment of the beam.

In this paper, steel plates with low yield strength (F_y = 35 kip) and 29000 ksi elastic modulus were used for the steel shear wall. The rehabilitated structure with the steel shear wall was modeled with a strip model. For the strip model, the trilinear force-deformation relationship with 2 % strain hardening was used, which was derived from the work of Purba and Bruneau [31]. Fig. 4–5 show the rehabilitated structure with the steel shear wall and the used force-deformation relationship, respectively.

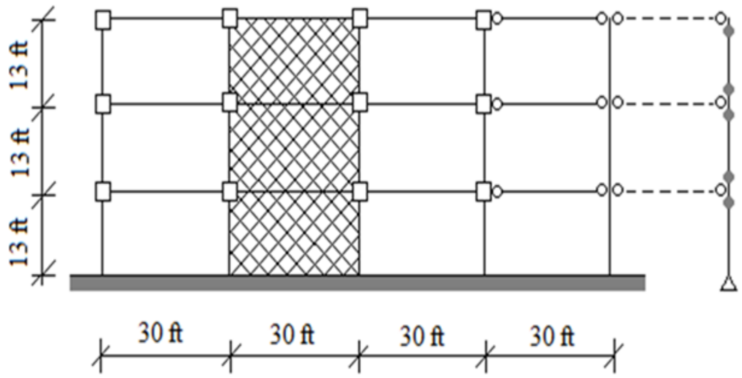

Figure 4. Rehabilitated structure with steel shear wall.

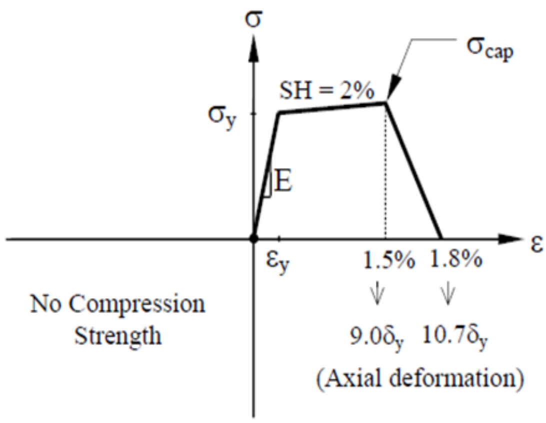

Figure 5. Trilinear force-deformation relationship of steel shear wall [31].

2.3. Selected records to evaluate the seismic performance

The incremental dynamic analysis method describes the behavior of the structure in a wide range of different intensities of the earthquake. In the incremental dynamic analysis method, the influence of different records is considered with a deterministic model of the structure; hence, this method involves only the aleatory uncertainty. A developed dynamic analysis method is proposed to consider epistemic uncertainties. In this method, aleatory and epistemic uncertainties are considered by using the probabilistic distribution of the structural model. Records were selected and scaled according to the location of the structure to perform incremental dynamic analysis. Selected records include far-field and near-field records. So far, extensive studies have been done to define the near-field record and to distinguish it from a far-field record. In the classification of records, the distance from the site to the earthquake center is the distinctive feature between far-field and near-field records. Usually, in near-field records, distances between 20 and 60 km are defined as near faults. However, there is no global definition of which site may be classified as a near or far fault. In this paper, 15 km distance from the fault was considered as a criterion for the classification of far-field and near-field records [32]. All selected records are located on D soil [33]. Tables 2–3 show far-field and near-field records.

Table 2. Details of the far-field records.

No.	Earthquake	Year	Station	M_W	R_{ib} (km)	R_{rup} (km)
1	San Fernando	1971	2516 Via Tejon PV	6.61	55.2	55.2
2	Tabas, Iran	1978	Ferdows	7.35	89.76	91.14
3	Imperial Valley06	1979	Coachella Canal #4	6.53	49.1	50.1
4	Victoria, Mexico	1980	SAHOP Casa Flores	6.33	39.1	39.3
5	Coalinga-01	1983	Parkfield–Cholame	6.36	55.05	55.77
6	N. Palm Springs	1986	Hesperia	6.06	71.7	72.97
7	Whittier Narrows-01	1987	Canyon Country-W Lost	6	44.88	48.18
8	Loma Prieta	1989	Richmond City Hall	6.93	87.78	87.87
9	Landers	1992	Baker Fire Station	7.28	87.94	87.94
10	Northridge-01	1994	Huntington Bch-Waikiki	6.69	66.43	69.5

Table 3. Details of the near-field records.

No.	Earthquake	Year	Station	M_W	R_{ib} (km)	R_{rup} (km)
1	Parkfield	1966	Cholame–Shandon	6.19	12.9	12.9
2	Gazli	1976	Karakyr	6.80	3.92	5.46
3	Coalinga	1983	Pleasant Valley P.P. bldg	6.36	7.69	8.41
4	N. Palm Springs	1986	North Palm Springs	6.06	0	4.04
5	Whittier Narrows-01	1987	Santa Fe Springs E. Joslin	6	11.47	14.49
6	Superstition Hills-02	1987	Parachute Test Site	6.54	0.95	0.95
7	Loma Prieta	1989	Gilroy Araay #2	6.93	10.38	11.07
8	Erzican, Turkey	1992	Erzican	6.69	0	4.38
9	Kobe	1995	KJMA	6.9	0.94	0.96
10	Chi-Chi	1999	TCU065	7.62	0.57	0.57

The selected records were scaled according to NIST GCR 11-917-15 publication [34]. In two-dimensional analyses, the selected records should be scaled so that the mean value of the response spectrum for the set of records is not lower than the site design response spectrum within the periodic range of 0.2*T* to 1.5*T*. In Fig. 6, the site design response spectrum, the scaled spectrum of the selected records, and the mean spectrum of these records are indicated.

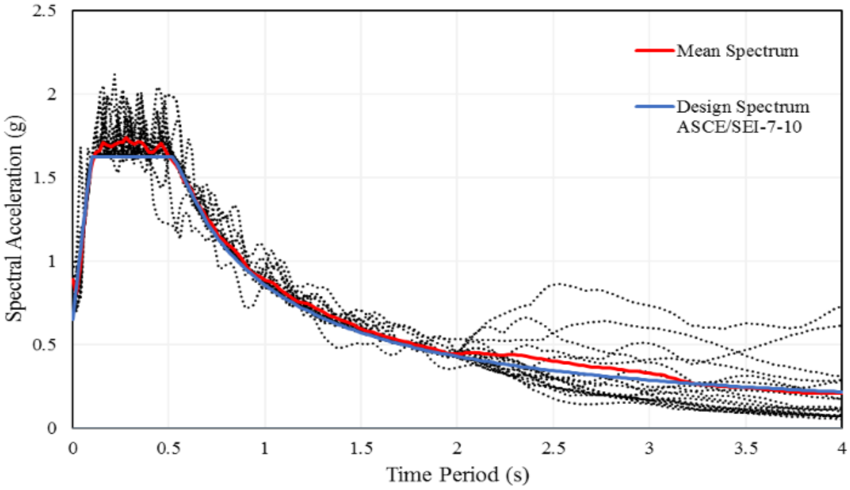

Figure 6. The design spectrum and the scaled spectrum of selected records along with the average range of these records.

2.4. Evaluation of probabilistic seismic performance

Despite large uncertainties in the seismic demand and capacity, the results of the design and evaluation of the seismic performance of structures is valid only if these uncertainties are considered with a realistic approach. In other words, designing and evaluating the seismic performance of structures should be done using reliability methods. Reliability was first used by researchers who were working in the field of structural safety. The purpose of reliability is to express the completeness of probability of failure [35]. Each reliability issue has two components: random variables and state functions. Random variables express uncertainty in the problem while the state functions define the failure event. In general, the limit state function is defined as Eq. (7):

$$g\left(R.S\right)=R-S, \tag{7}$$

where g is a limit state function, and a boundary limit state occurs in the state of $g=0$. In this equation, $g>0$ indicates a safe structure, and $g<0$ denotes the failure of the structure. R and S express the capacity and demand, respectively, which include a set of random variables. The probability of failure $\left(P_f\right)$ is expressed in Eq. (8) as the multiple integrals:

$$P_f = P(g \le 0) = \int_{g\le 0}\int f(x)dx, \tag{8}$$

where $f(x)$ is the probability density function (PDF) for random variables. Variations of statistical distribution for the randomized model and number of random variables can complicate the evaluation of Eq. (8) using the integration method. Different methods of reliability are used to solve this problem. In this paper, the Latin Hypercube Sampling (LHS) method was used to consider demand and capacity uncertainties of the structural system, which is a common example of reliability methods.

Initially, the structure with basic parameters was analyzed with selected seismic records. The results of this stage were associated with uncertainty caused by various seismic records. In the next step, different realities of the structural model were formed considering force-deformation relationship parameters of the steel shear wall as probabilistic variables. Epistemic uncertainty was determined by conducting incremental dynamic analysis on each of these structural realities. Table 4 shows the statistical characteristics of the selected probabilistic variables. 160 structural realizations were used based on the considered probabilistic variables for the rehabilitated structure with the steel shear wall [36].

Table 4. Statistical characteristics of random input variables.

Name	Symbol	Mean	C.O.V	Distribution	Reference
Steel yield strength	F_y	35 kip	0.07	Lognormal	Melchers [37], JCSS [38]
Elastic stiffness	E	29000 ksi	0.03	Lognormal	Schmidt and Bartlett [39], Dexter et al. [40]
Strain hardening	a_h	0.02	0.4	Normal	Sadowski et al. [41–42]
Post-capping stiffness	a_c	–0.68	0.4	Normal	Lignos and Krawinkler [43]

According to FEMA 356 [44], the inter-story drift ratio is considered an index to measure of structural damage. Values of 0.7 %, 2.5 %, and 5 % were proposed by FEMA 356 as the permissible inter-story drift ratio for three structural performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), respectively. In this paper, the spatial acceleration in the main period of structure $\left(S_a\right)$ was used to measure the seismic intensity. The fragility function $F_r(x)$ was also used to express the probability of exceedance of different damage states, which is expressed by Eq. (9) [45]:

$$F_r(x) = \phi\left[\left(\ln x - \ln \widehat{S}_a\right)\middle/\beta_R\right], \tag{9}$$

where $\widehat{S}_a$ expresses the median value of the structural fragility in unit S_a, β_R is the standard deviation of lognormal of the system fragility, and Φ denotes the standard normalized cumulative distribution function. The uncertainties associated with seismic demand and the structural capacity are expressed by the β_R parameter that is calculated using Eq. (10):

$$\beta_R = \sqrt{\beta^2_{\mathrm{D}|S_a} + \beta^2_c}. \tag{10}$$

$\beta_{\mathrm{D}|S_a}$ and β_c parameters represent the uncertainty in seismic demand and the uncertainty in the structural capacity, respectively. In limit states of IO and LS, the value of β_c is 0.25, and in the limit state of CP is 0.15 [45–46]. The annual exceedance probability of the limit state is defined in Eq. (11):

$$P_{LS} = k_0 \widehat{S}_a^{-k} \exp\left[\frac{(k\beta_R)^2}{2}\right], \tag{11}$$

where $k_0 \widehat{S}_a^{-k}$ expresses the seismic hazard, and the exponential expression is the correction factor that applies to the variability of seismic demand and structural capacity; k_0 and k indicate the risk scale and the slope of the seismic hazard curve, respectively.

3. Results and Discussion

3.1. Sensitivity analysis

In this paper, the sensitivity of the rehabilitated structure response with the steel shear wall was evaluated to assume yield strength, elastic stiffness, strain hardening, and post-capping stiffness of the trilinear force-deformation relationship of the steel shear wall as the probabilistic parameters. For each of these parameters, base value and an appropriate range of changes were considered. In Fig. 7, the IDA curves show the sensitivity of the rehabilitated structure response with the steel shear wall to the probabilistic parameters.

The values of 30, 35 and 40 ksi were considered for the yield strength parameter to investigate the effect of the sensitivity of the structure response to this parameter. The value of 35 ksi was considered as the base value, and the values of 30 and 40 ksi were considered as the lower and upper bounds of the selected range for this parameter, respectively. In Fig. 7a, the IDA curves show the sensitivity of the rehabilitated structure response with the steel shear wall to the yield strength parameter. The values of 25000, 29000, and 33000 kip were considered for the elastic stiffness parameter with 29000 kip considered as the base value. The IDA curves in Fig. 7b show the sensitivity of the response of the rehabilitated structure with the steel shear wall to this parameter. As can be seen in Fig. 7a-b, the effect of the two probabilistic parameters of yield strength and elastic stiffness of trilinear force-deformation relationship is more on the response of the rehabilitated structure with steel shear wall (17.5 % and 19 %).

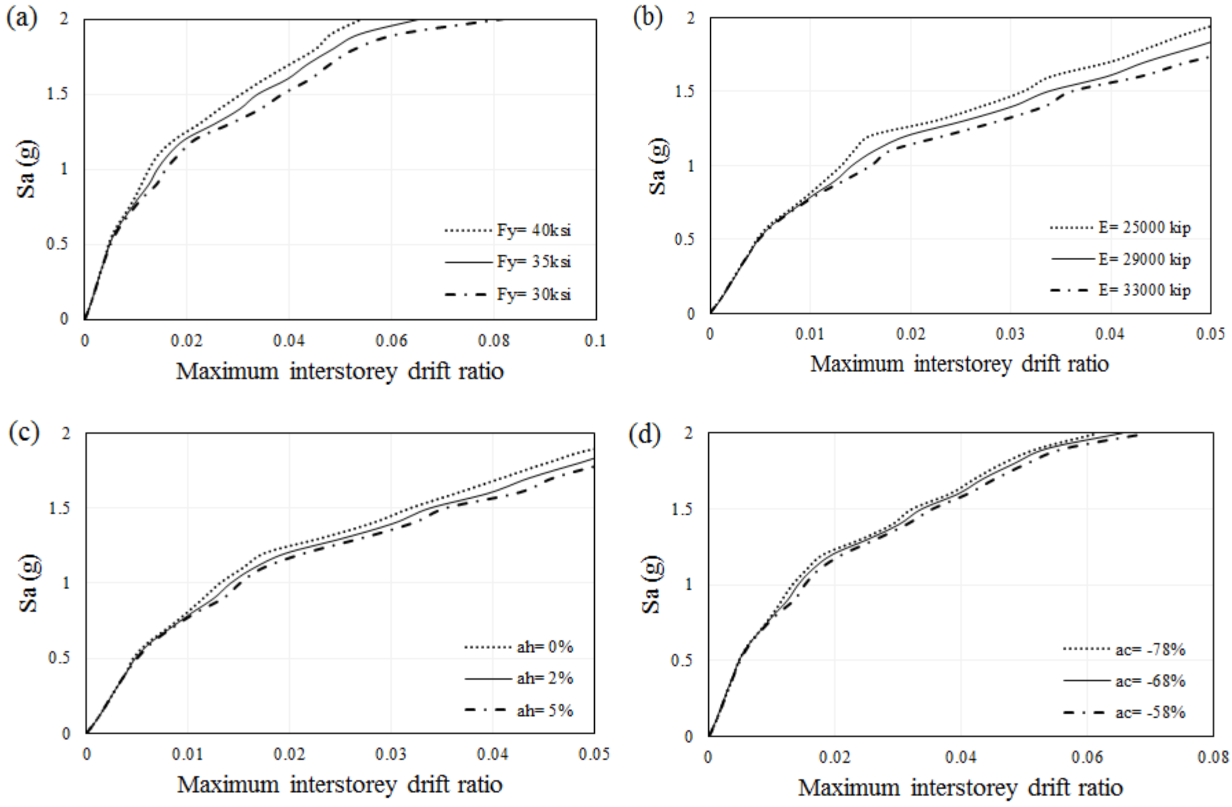

Figure 7. The sensitivity of median IDA curve to probabilistic parameters of steel shear wall: (a) The effect of steel strength parameter, (b) The effect of elastic stiffness parameter, (c) The effect of strain hardening parameter, (d) The effect of post-capping stiffness parameter.

The strain hardening is defined as a ratio of the plastic stiffness to the elastic stiffness in the force-deformation relationship of the steel shear wall. The value of 2 % was considered as the base value, and the values of 0 % and 5 % were considered as the lower and upper bounds of the selected range for this parameter, respectively. Fig. 7c shows the IDA curves of rehabilitated structure with the steel shear wall for base value and values of the considered range. Based on the results, the effect of this parameter on the response of the rehabilitated structure with steel shear wall is 7.6 %.

The post-capping stiffness, called negative strain hardening, was considered as probabilistic parameter with a base value of –68 % and a range of –78 % to –58 %. The sensitivity of the rehabilitated structure response to this parameter was indicated in Fig. 7d. As can be seen, the effect of the post-capping stiffness parameter of the trilinear force-deformation relationship of steel shear wall in the rehabilitated structure with the steel shear wall is 6.1 % that is less than the three considered probabilistic parameters.

3.2. Quantifying the considered uncertainties

Structural fragility curves indicate the exceedance probability of the defined damage states for records with specific intensity. Here, the maximum inter-story drift ratio was used to define the damage states, and values of 0.7 %, 2.5 %, and 5 % were used for Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) as performance levels of FEMA. In Fig. 8–11, structural fragility curves and IDA curves are shown for the structure before and after the rehabilitation and with considering uncertainties.

In Tables 5–6, values of uncertainties of seismic demand and total uncertainties of the structure, which are shown by parameters $\beta_{\mathrm{D}|S_a}$ and β_R, were included for the structure before and after of rehabilitation. In these tables, the median values of fragility were calculated according to Eq. (9) for the structure before and after rehabilitation and with considering uncertainties.

According the results in the Tables 5–6, the dispersion of the aleatory uncertainty of the steel moment frame structure was 0.32 and 0.38 for far-field and near-field records, respectively. This value was close to the dispersion value of steel moment frame structure [47]. The dispersion of the aleatory and epistemic uncertainties of the rehabilitated structure with steel shear wall was 0.27–0.34. This value was lower than the dispersion value of moment frame equipped with brace [47], and it was more than the dispersion value of moment frame structure [14].

As can be seen in Tables 5–6, the median value of fragility increased due to rehabilitation of the structure in the limit state of LS under far-field records from 0.47 g to 1.27 g and under near-field records from 0.57 g to 1.49 g. Also, it can be seen, median values of fragility increased due to rehabilitation of the structure in other limit states. The results of fragility curves of the rehabilitated structure with and without considering uncertainties showed that median values of fragility were reduced by considering uncertainties in all limit states.

The results of the IDA curves showed that rehabilitation of the structure reduced the maximum inter-story drift of the structure in the limit states of IO, LS, and CP. Comparison of the IDA curves of the rehabilitated structure with and without considering the uncertainties demonstrated that the maximum inter-story drift of the structure increased by 28.8–52 % under far-field records and by 24.1–39.3 % under near-field records by means of considering the uncertainties. This shows that conservatism was reduced by considering the uncertainties.

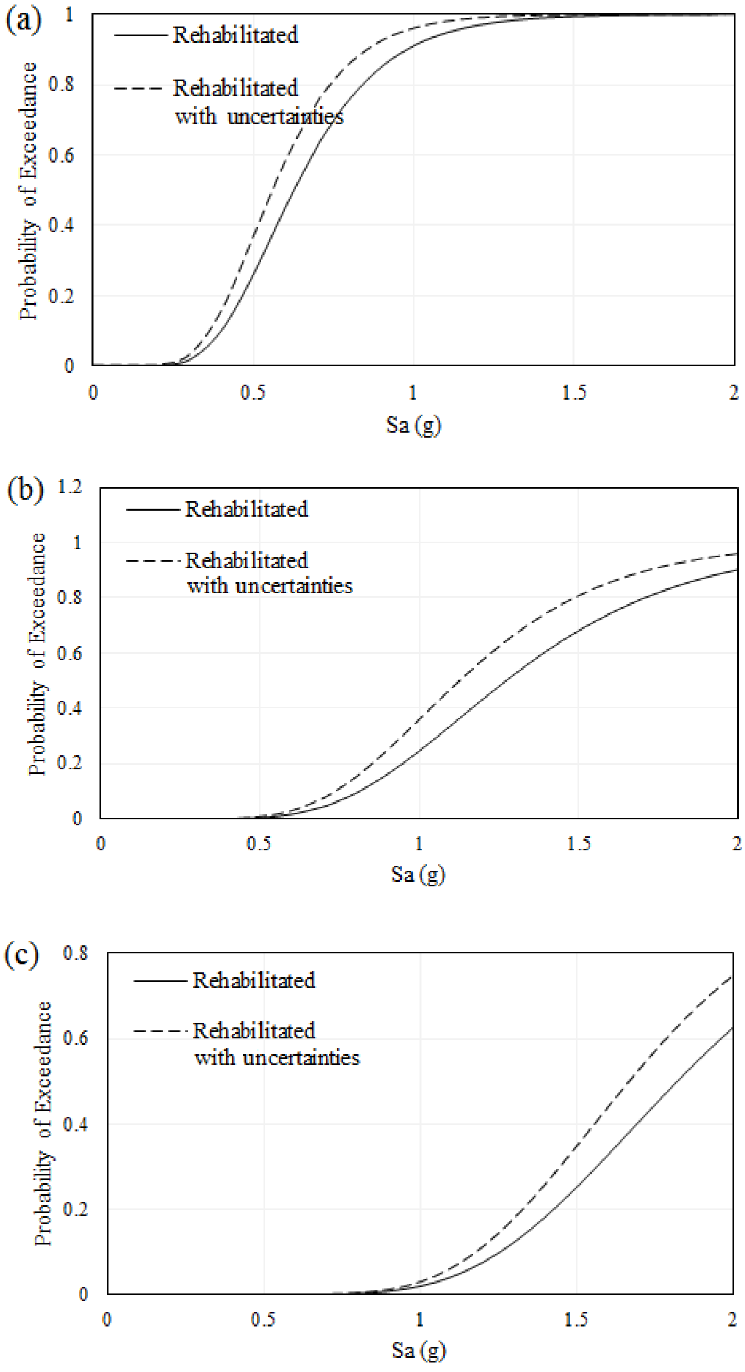

Figure 8. Fragility curve for the limit states under far-field records: (a) the limit state of IO, (b) the limit state of LS, (c) the limit state of the CP.

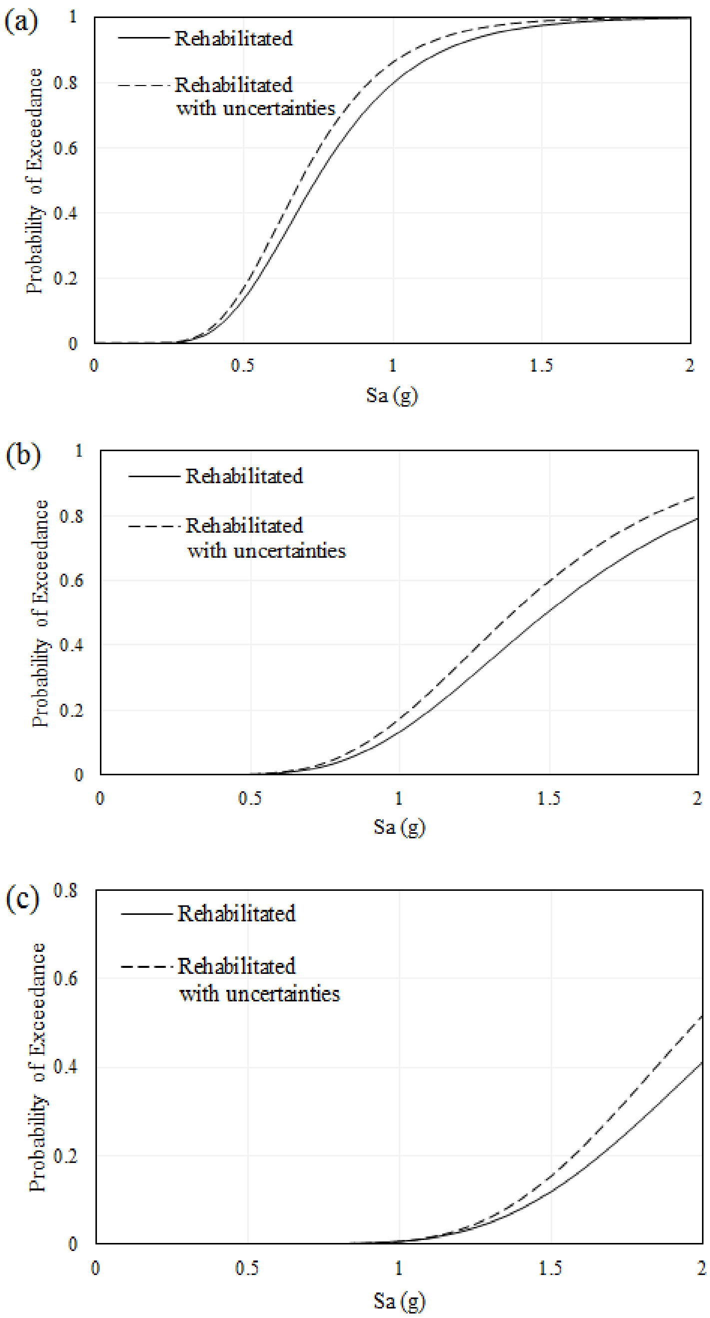

Figure 9. Fragility curve for the limit states under near -field records: (a) the limit state of IO, (b) the limit state of LS, (c) the limit state of the CP.

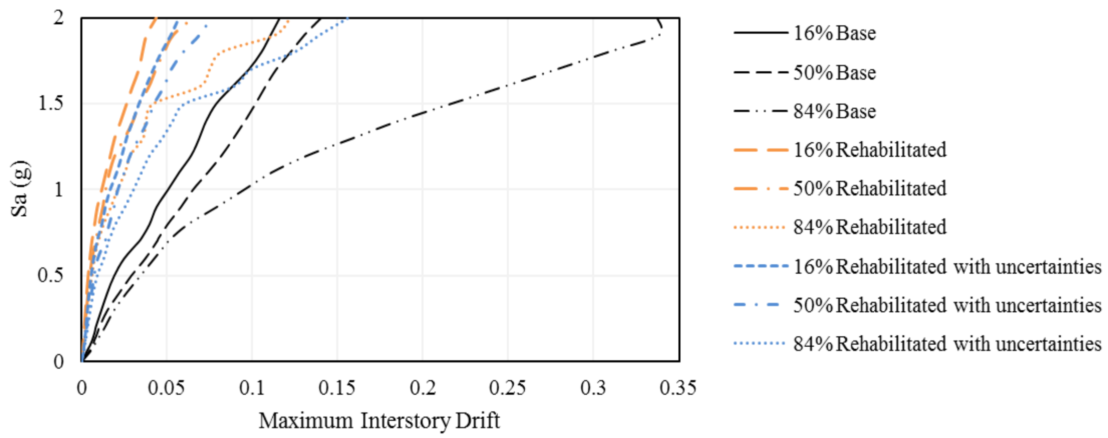

Figure 10. IDA curve of the structure under far-field records.

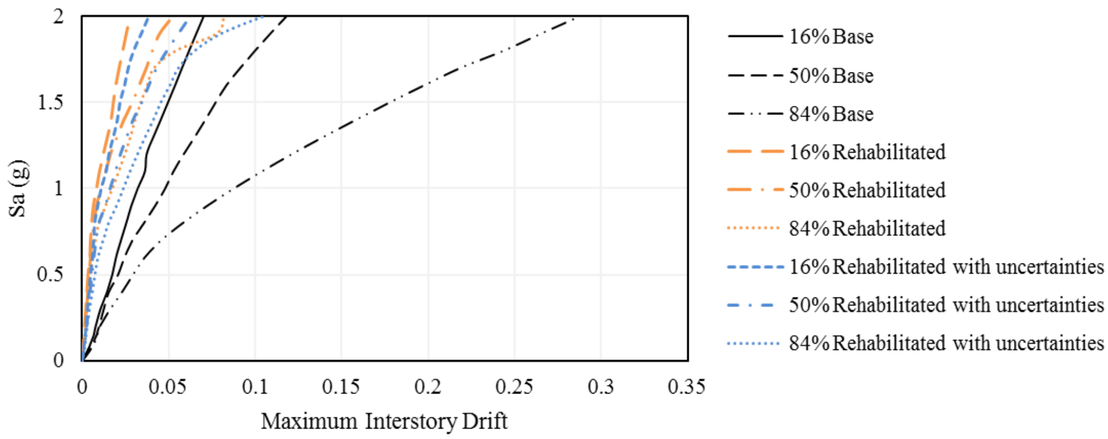

Figure 11. IDA curve of the structure under near-field records.

Table 5. Seismic demand statistics and median fragility values of structures under far-field records.

Case	$\beta_{D\|S_a}$	β_R			$\hat{S}_a(g)$		
		IO	LS	CP	IO	LS	CP
Base building	0.32	0.41	0.41	0.36	0.11	0.47	0.79
Rehabilitated building	0.25	0.35	0.35	0.29	0.63	1.27	1.82
Rehabilitated building with uncertainties	0.22	0.33	0.33	0.27	0.56	1.13	1.67

Table 6. Seismic demand statistics and median fragility values of structures under near-field records.

Case	$\beta_{D\|S_a}$	β_R			$\hat{S}_a(g)$		
		IO	LS	CP	IO	LS	CP
Base building	0.38	0.45	0.45	0.41	0.13	0.57	1
Rehabilitated building	0.26	0.36	0.36	0.30	0.74	1.49	2.14
Rehabilitated building with uncertainties	0.23	0.34	0.34	0.27	0.69	1.38	1.97

3.3. Annual exceedance probability of the limit states

The annual exceedance probability of three limit states of IO, LS, and CP was calculated for the structure before and after the rehabilitation and considering the uncertainties based on Eq. (11). In this equation, k_0 and k are the seismic hazard parameters, which are considered 3.03e-4 and 2.69 for the Los Angeles region [47]. The results of the calculated annual exceedance probability are presented in Tables 7–8. The annual exceedance probability value was 0.000064–0.0021 for rehabilitated structure with considered uncertainties, and it was lower than the value of moment frame equipped with brace [47].

Table 7. Annual exceedance probability of selected limit states under far-field records.

Case	**Annual exceedance probability (P_{LS})**		
	IO	LS	CP
Base building	0.2109	0.00424	0.000913
Rehabilitated building	0.0016	0.00025	0.000082
Rehabilitated building with uncertainties	0.0021	0.00032	0.000099

Table 8. Annual exceedance probability of selected limit states under near-field records.

Case	Annual exceedance probability (P_{LS})		
	IO	**LS**	**CP**
Base building	0.1524	0.00286	0.000557
Rehabilitated building	0.0011	0.00017	0.000048
Rehabilitated building with uncertainties	0.0013	0.00020	0.000064

As the Tables 7–8 show, the annual exceedance probability of three limit states was reduced with the rehabilitation of the structure. This shows that rehabilitation of the structure improved the seismic performance of the structure. In addition, the results of the Tables 7–8 show that the annual exceedance probability of the limit states of IO, LS, and CP increased by 31.2, 28 and 20.7 % under far-field records, and also this parameter increased by 18.2, 17.6 and 33.3 % under near-field records by considering the uncertainties in the rehabilitated structure. Comparing the results of the Tables 7–8 showed that reduction or growth in annual exceedance probability under far-field records was greater than near-field records. Based on the response spectrum of selected far-field records, this may be due to the greater impact of these records in the main period of the rehabilitated structure.

4. Conclusions

In this paper, a 3-story structure of the SAC project was selected, verified in OpenSees software, and rehabilitated with a steel shear wall. The structure was analyzed with a base model and with selected records. The results of this case captured the uncertainty caused by various seismic records. In this paper, the trilinear force-deformation relationship was considered for the steel shear wall. Based on this relationship, four parameters of yield strength, elastic stiffness, strain hardening, and post-capping stiffness were considered as probabilistic variables, and probabilistic models of the structure were created. Both aleatory and epistemic uncertainties were considered taking into account probabilistic models of the structure and with performing incremental dynamic analysis.

1. Sensitivity analysis was done in order to investigate the effect of each considered probabilistic variable on the structural response. The results of the sensitivity analysis showed that the effect of the two parameters of the yield strength and elastic stiffness of the force-deformation relationship of the steel shear wall on the response of the rehabilitated structure with steel shear wall are greater (17.5 % and 19 % respectively), while the effect of the post-capping stiffness parameter is the lowest (6.1 %).

2. The results of the analysis showed that rehabilitation of the structure reduced the maximum inter-story drift of the structure in three limit states of Immediate Occupancy, Life Safety, and Collapse Prevention. Comparison of the IDA curves of the rehabilitated structure with and without considering the uncertainties demonstrated that the maximum inter-story drift of the structure increased by 24.1–52 % by means of considering the uncertainties. This shows that considering the uncertainties reduced conservatism.

3. The annual exceedance probability of three limit states of Immediate Occupancy, Life Safety, and Collapse Prevention was reduced by the rehabilitation of the structure. This showed that rehabilitation of the structure improved the seismic performance of the structure. In addition, the results showed that the annual exceedance probability of three considered limit states by considering uncertainties in the rehabilitated structure increased by 20.7–31.2 % under far-field and 17.6–33.3 % near-field records, respectively. The analysis results indicated that far-field records have more influence on the results due to the greater impact of far-field records in the main period of the rehabilitated structure.

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Received 04.03.2020. Approved after reviewing 28.09.2022. Accepted 21.11.2022.