

Original Research Paper

# Dynamic Reliability Analysis of Steel Moment Frames Using Monte Carlo Technique

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## Article history

Received: 7-02-2019

Revised: 16-03-2019

Accepted: 15-06-2019

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**Abstract:** Designing of structures is the function of operation of them. However, operation of the structures depends on different uncertainties. In practical cases, uncertain parameters are simplified and formulated to certain parameters which are implemented in the designing approach. In this investigation, different structural parameters which bear with uncertainties including loading conditions, strength of materials and geometry of the structural elements are modeled and simulated based on Monte Carlo technique. Nonlinear dynamic time history analyses are conducted for different structures with steel moment frames based on finite element method using OpenSees code. Different responses are monitored for different accelerograms for both far and near field faults. Based on the results, the reliability index of steel frames in far field cases are larger than in near field cases.

**Keywords:** Dynamic Reliability Analysis, Monte Carlo Simulation, Steel Moment Resisting Frame SMRF

## Introduction

Safety analysis of structures in seismic conditions is one of the significant phases of structural design especially in urban areas with high hazard of earthquake. A review of case histories shows that in some cases even in buildings with conventional seismic design, severe earthquake failures were monitored which coincide with severe economic losses and casualties. It is noteworthy that conventional seismic design procedures are developed to reduce structural failure severity and increase overall safety of the inhabitants in regular earthquakes. However, typical seismic design of structures cannot provide absolute safety during intense earthquakes and prevent drastic disasters completely (Rezaian, 2014).

In the last two decades, theory of structural reliability against earthquakes has been discussed and developed by many investigators and can be considered as one of the most fundamental designing aspects of recent structures. In the practical viewpoint, reliability analysis has been conducted for many structures all around the world and the effectiveness of the procedure has been verified and reported by different designers and researchers (Jahani, 2012). Advancing of computer programs and decrease of

computing time provide suitable infrastructure to apply more uncertainty parameters in seismic design of structures. Consequently, consideration of various uncertainty parameters enhances the quality and effectiveness of seismic design procedure. For a high quality seismic design of structures, it is expected that different seismic risks and structural and loading complexities would be considered. Neglecting of any considerable variable in seismic design may cause severe disaster during the operation of structures within earthquake occurrence and also after the coincidence. Therefore, it is expected that constructing buildings especially in urban areas should be based on consideration of reliability and safety of the structures against earthquakes. One of the most important problems that must be considered is the level of reliability and safety during the lifetime of a system. This issue can be solved using theories and tools for structural reliability assessment. Based on theories of structural reliability, one can use the theory of probability to convert structural parameters with inherent uncertainty to mathematical correlations. Therefore, safety considerations of structural operation can be embedded into seismic design of structures using quantitative data (Hong *et al.*, 2010).

It is noteworthy that evaluation of structural reliability is not just a contemporary concern among

engineers. There has been many efforts to find the seismic risk of structures since (Datta, 2010; Jahani *et al.*, 2013). In traditional methods, the reliability of structures is determined based on applying safety coefficients in their seismic design (Jahani, 2012). These safety coefficients have been assigned usually with a same weight to the designing correlations to consider different random effective parameters in structures including loading conditions, materials, geometry and boundary conditions. Applying coefficients provides a proportional guarantee for structures for not facing failure or reaching to threshold stresses or allowable deformations. This methodology is based on the preliminary structural guideline and approximate solutions of structural analyses (Bojórquez *et al.*, 2008; Watstein, 1953).

Applying random effective parameters in structures based on reliability theories have been considered since 1970s (Jahani, 2012) has been recognized as one of the most appropriate and robust solutions to predict the functionality of structures with acceptable procession in practical applications (Moosavi, 2012). Using nonlinear time history analysis is seemed to be one of the most appropriate methods for simulation of earthquake effects on structures (Jankowiak and Tlodygowski, 2005). However, this method has also its own disadvantages including being a time consuming and costly method for conventional designs, technical complexities for providing necessary input parameters, lack of sufficient statistical data for providing equivalent records for different circumstances (Sayyar-Roudsari *et al.*, 2019; 2018).

## Reliability Analysis

In the recent century, the theory of reliability analysis has been developed considerably. The first period of the theory development can be considered from 1920 to 1960. In this period, different investigations developed different aspects of theory of reliability analysis (Freudenthal, 1947; Mayer, 1926; Weibull, 1939). Other investigators developed various concepts of structural random incidences (Basler, 1960; Johnson, 1953; Plum, 1950). However, in practical designing methods, these developments have not been used until 1960s. Conventional designing procedures were based on other investigations in the area of linear elastic methods, plastic methods, structural analysis and mathematical formulation of especial structures such as shell systems. In these investigations, although forces and strength parameters were considered as uncertain parameters, extreme maximum and minimum limits were assumed for their threshold and safety coefficient were applied for considering differences in construction methods. However, even safety coefficients were assumed based on engineering judgment. One of the most

important parameters in safety zones of structures were relative distribution of random variables.

Mayer in Germany (1926) and later Basler in Switzerland (1960) provided specific measurement for explaining reliability of structures using Cornell reliability index based on second moment (Cornell, 1969). Forssell (1924) explained optimization criterion for structures as a design which minimize total required cost including costs for initial construction cost and expected costs due to the structural failure.

Based on Mayer investigations, the design of structures would be conducted considering average and standard deviation of random variables. Plum (1950) evaluated the differences between economical safety limits and failure rates in reinforced concrete slabs. Johnson (1953) provided a comprehensive solution based on structural reliability and optimal design theories including statistical theories of developed resistance which had been suggested by Weibull (1939). In another study by Freudenthal (1947), primary reliability of structures under accidental loading conditions were discussed. The results of this study were used by a wide range of engineers. Between "1967-1974" structural reliability issues were taken into consideration by many investigators including Freudenthal (1947) Ferry-Borges and Castanheta (1971), Pugsley (1966), Johnson (1953) and Turkstra (1970). These investigators provided designing criteria for structures considering different uncertainty parameters and risk factors.

Generally, uncertainty parameters can be categorized in three different aspects including: (1) accidental uncertainties due to variation of excitation, (2) statistical uncertainties due to estimation of different parameters describing a statistical model and (3) uncertainties which are based on mathematical simplification of complicated physical phenomenon. Some examples of mathematical simplifications are linear analysis of nonlinear materials and discrete modeling with limited degree of freedom for a continuous variables. In spite of accidental uncertain variables due to variation of excitation which cannot be reduced, variables based on the last two categories can be modified and decreased. For instance, a comprehensive collection of data or samples can reduce uncertainties due to statistical issues. Similarly, detailed modified mathematical model can reduce uncertain variables of a simplified mathematical solution. Therefore, conducting a more complicated model with a thorough database can decrease uncertainties.

In dynamic reliability analysis of structures, uncertain parameters of earthquake can affect the estimation of reliability solution considerably. Therefore, dynamic uncertainty parameters are taken into more consideration than other uncertainty parameters. In practice, seismic reliability analysis of structures are based on considering

random ground motions, inherent uncertainty of earthquake happenings and parameters related to earthquake amplitude. It is noteworthy that some investigators combine both seismic random variables and physical and strength of material uncertainties in a specific designing procedure (Fallahi *et al.*, 2018). However, considering both series of uncertainties can make the analysis too complicated and impractical.

Probability models which are widely used for describing the distribution of uncertainty parameters are including uniform, exponential, normal lag and Poisson's distributions. There would be several different dynamic reliability methods based on considering the type of the distribution. Moreover, material properties and other uncertain parameters may affect analytical methods. Using approximate correlations for material properties can reduce the complexity of solutions (Datta, 2010).

### Formulation of Reliability Analysis

Different formulation method can be used for reliability analyses. Two different categories can be mentioned for the formulation methods including constant time and variable time problems. In each of these two categories, a limit state function can be defined based on different criteria including functionality, ultimate stress, etc. The purpose of reliability analysis is to find the probability of limit state failure or violation of limit state conditions. For problems with constant time,  $g(x) = 0$  defines the limit state function where  $x$  is a collection of random variables including  $x_1, x_2, \dots, x_n$ . In this function,  $g(x) \leq 0$  shows the failure status. Therefore, the probability of failure is defined as following equation (Datta, 2010):

$$P_f = p[g(x) \leq 0] = \int_D f(x) dx \quad (1)$$

where,  $f(x)$  is the density function of jointed probability for  $g(x) = 0$  and integration of domain of  $D$  shows the location of  $g(x) \leq 0$ .

Figure 1 shows the mentioned concept for two special random variables. The integration of an appropriate probability density function is the hatched zone shows the reliability capability of the structure against failure in a limit state of  $g(x_1, x_2) = 0$ . A time dependent reliability problem is in a case that the limit state function would be dependent to time.

In this case the limit state function is defined based on  $g(x, y(t))$ ; where  $x$  is a set of random variables and  $y(t)$  is a time dependent vector of random process. In these problems the failure status is passing of vector process of  $y(t)$  through limit state surface of  $g(x, y) = 0$  as shown in Fig. 2. Therefore, we have (Datta, 2010):

$$P_f = \int p[\min g(x, y(t)) \leq 0 | x] f(x) dx \quad (2)$$

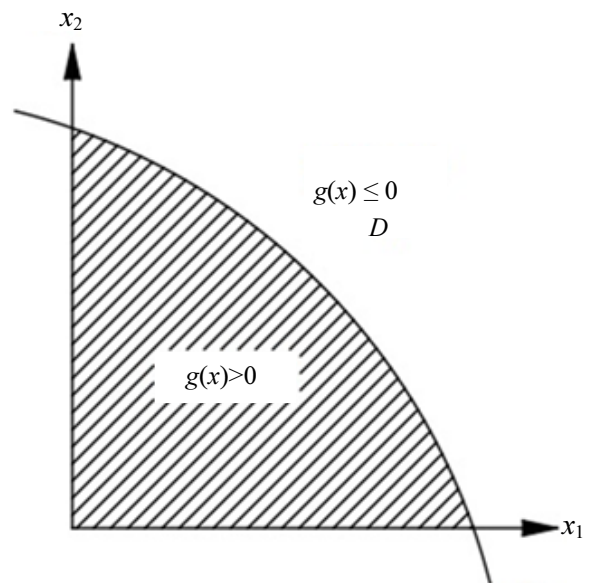


Fig. 1: Reliability capability concept with two random variables (Datta, 2010)

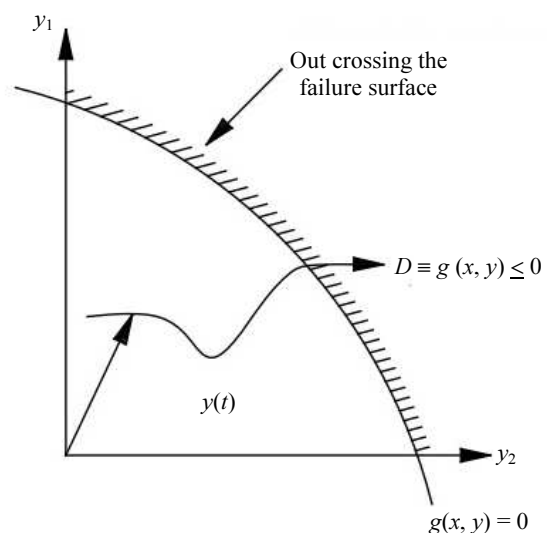


Fig. 2: Reliability capability problem with time variable (Datta, 2010)

where,  $T$  is the lifetime of the structure and  $f(x)$  is jointed probability density function. In Equation (2), the conditional probability of the phrase in the bracket can be evaluated by a statistical analysis. An accurate solution for Equation (2) would be complicated. This is because of complexity of evaluation of the conditional probability of the phrase in the bracket and also determining the jointed probability of dense function  $f(x)$ .

The second item is a reason for inability for acquiring an exact solution for integral of Equation (2). Therefore,

there is a wide range of approximation and assumption for analyzing the reliability capability based on Equation (2). However, in different reliability studies, adequate assumptions have been assumed to evaluate seismic reliability capability of structures (Datta, 2010).

The threshold crossing reliability analysis is the out-crossing analysis in which exceedance probability of a threshold value of the response is determined. The threshold crossing reliability of the structure is defined as one minus the value of the threshold crossing probability. Considering a frame that has uncertain properties represented by the uncertainties of the EI values of beams and columns, the time history of excitation is deterministic. The problem is formulated as a time variant reliability problem with a limit state function defined as:

$$g(x,t) = u_0 - u(x,t) \quad (3)$$

where,  $u_0$  is the threshold value of the top displacement of a frame and  $u(x,t)$  is the top-story displacement of the frame at a given time ( $t$ ). The  $u(x,t)$  is a function of a number of random variables given by the vector  $x$ . Assuming the random variables to be independent and normally distributed variables and the Taylor series expansion of the limit state function to be truncated at the first-order term, the mean and variance of  $g(x,t)$  can be determined.

### Model Structures

In this study, five models of steel moment resisting frame with 3, 5, 7, 10 and 15 floors are selected. It is assumed that connections are rigid in all of the models. Spans of frames are 5 m and the height of each floor is 3.2 m. According to ASCE/SEI 41-13 (ASCE, 2017), it is

assumed that structures are constructed in area with high hazard of earthquake with soil type of III. Figure 3 and 4 and Table 1 show the detail of structures. As shown in Fig. 3 All of frames have three spans. It is assumed that structures are residential buildings with medium level of significance.

The static loading of structures are based on standard guideline of ASCE/SEI 41-13 (ASCE, 2017). The dead and live load for all of stores except the roof level are 2500 and 1000 kg/m, respectively. The roof dead load for models with 3 and 5 floors are 1750 kg/m, for models with 7 and 10 floors are 2083 kg/m and for 15 floor model is 2250 kg/m. Live load for all models are assumed as 750 kg/m. For the study, the degree of significance is assumed as 1, behavior coefficient is 0 and the Peak Ground Acceleration (PGA) assumed as 0.35 g. Effects of P-Δ has been considered for designing structures. For structural design, ETABS version 2013 (CSI, 2013) and code of UBC97-ASD (UBC, 1997) have been used. It is noteworthy, that for simplicity, the interaction between soil and structures is neglected.

Yielding stress ( $f_y$ ) of the steel material used in the analysis is 2400 kg/cm<sup>2</sup>. Also, the modulus of elasticity (E) of the steel is considered as  $2.1 \times 10^6$  kg/cm<sup>2</sup> and second stiffness coefficient ( $\alpha$ ) of the steel is assumed as 0.05 (Gerami and Abdollahzadeh, 2015).

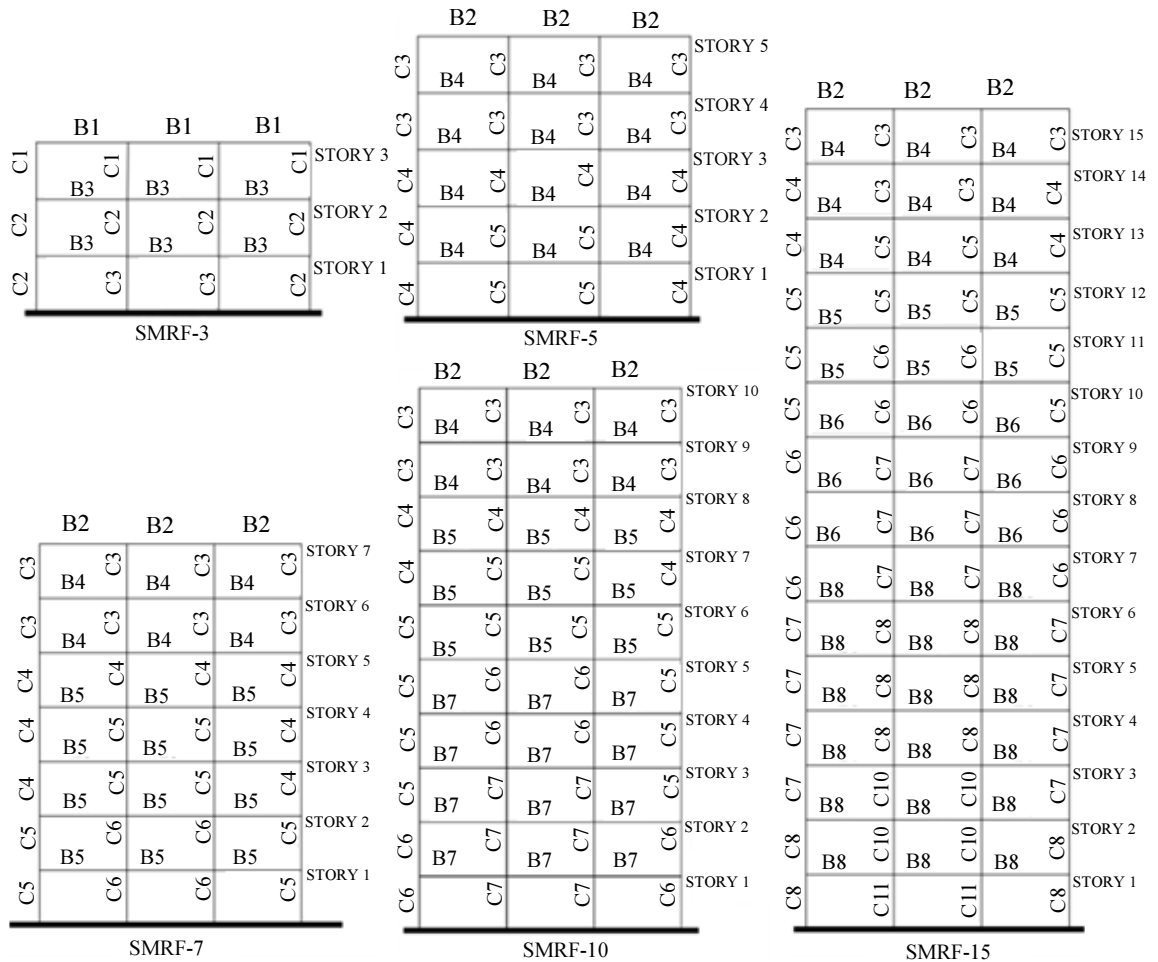
In this investigation earthquake records in the study of Gerami and Abdollahzadeh (2015) are used for evaluating the reliability capability of frames. Prosperities of these records are shown in Table (2). The time step for input accelerations are considered as 0.005 seconds. Based on ASCE/SEI 41-13 (ASCE, 2017), 5 percent of critical damping is assumed for seismic analysis of models. In near-field acceleration inputs both the maximum free-field acceleration and directional effects are more intense than far-field acceleration data.

**Table 1:** Geometry properties of structural sections

Section No.	Section name	B (cm)	t (cm)	H (cm)	S (cm)
B1	2PL150*12PL276*8	15	1.2	30	0.8
B2	2PL150*15PL270*8	15	1.5	30	0.8
B3	2PL200*15PL270*8	20	1.5	30	0.8
B4	2PL200*20PL260*8	20	2.0	30	0.8
B5	2PL200*20PL310*10	20	2.0	35	1.0
B6	2PL200*20PL360*10	20	2.0	40	1.0
B7	2PL250*20PL310*10	25	2.0	35	1.0
B8	2PL250*20PL360*10	25	2.0	40	1.0
C1	BOX150*150*10	15	1.0	15	1.0
C2	BOX200*200*12	20	1.2	20	1.2
C3	BOX200*200*15	20	1.5	20	1.5
C4	BOX200*200*20	20	2.0	20	2.0
C5	BOX250*250*20	25	2.0	25	2.0
C6	BOX250*250*25	25	2.5	25	2.5
C7	BOX300*300*25	30	2.5	30	2.5
C8	BOX300*300*30	30	3.0	30	3.0
C9	BOX350*350*20	35	2.0	35	2.0
C10	BOX350*350*25	35	2.5	35	2.5
C11	BOX350*350*30	35	3.0	35	3.0

**Table 2:** Earthquake records

No.	Earthquake	Year	Station	R (km)	PGA (g)	T <sub>p</sub> (sec)	T <sub>m</sub> (sec)
1	Chi-Chi, Taiwan	1999	CHY065	83.43	0.1	0.56	0.79
2	Chi-Chi, Taiwan	1999	TAP095	109.01	0.15	0.98	0.84
3	Loma Prieta	1989	CDMG58224	72.20	0.24	0.32	0.86
4	Loma Prieta	1989	CDMG58472	74.26	0.26	0.64	0.85
5	Kobe, Japan	1995	HIK	95.72	0.14	0.60	0.76
6	Loma Prieta	1989	CDMG58223	58.65	0.23	0.30	0.53
7	Manjil, Iran	1990	Qazvin	49.97	0.13	0.16	0.46
8	Northridge	1994	CDMG13122	82.32	0.10	0.38	0.44
9	Tabas, Iran	1978	Ferdows	91.14	0.10	0.24	0.29
10	Kocaeli, Turkey	1999	Bursa Tofas	60.43	0.10	0.68	0.93
11	Denali, Alaska	2002	Pump St. 10	2.74	0.32	0.94	1.52
12	Bam, Iran	2003	Bam	R<15	0.59	0.78	0.91
13	Chi-Chi, Taiwan	1999	CHY101	9.96	0.44	0.90	0.98
14	Chi-Chi, Taiwan	1999	TCU068	0.32	0.56	0.42	1.51
15	Imperial Valley	1979	CDMG 5158	1.35	0.43	0.24	1.31
16	Northridge	1994	DWP 75	5.19	0.49	0.22	0.72
17	Silakhor, Iran	2006	Chalan Cho.	R<15	0.45	1.52	1.82
18	Kocaeli, Turkey	1999	Y arimca	4.83	0.26	0.52	1.29
19	Zanjiran, Iran	1994	Meymand	R<15	0.42	1.36	1.73
20	Kobe, Japan	1995	Takatori	0.61	0.61	1.22	1.10



**Fig. 3:** Structural frames

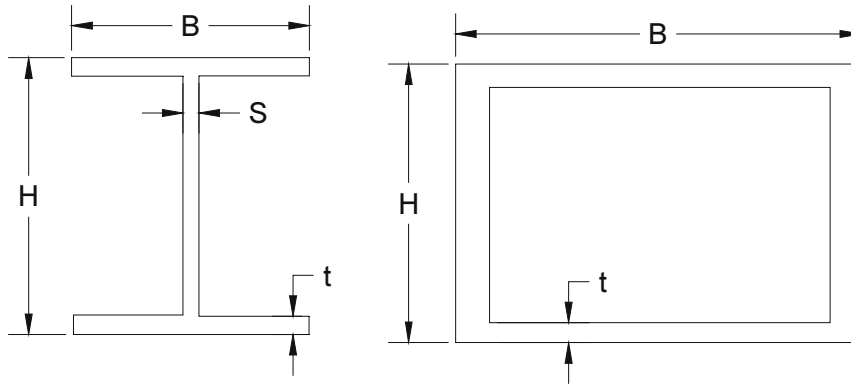


Fig. 4: Cross section of beams and columns (see Table 1 for values)

Table 3: Random variables of steel frames

Parameter	Distribution	Mean	Covariance
E	Lognormal	$2.1 \times 10^6$	0.1
$F_y$	Lognormal	2400	0.1
$\alpha$	Lognormal	0.05	0.1

This issue, increases the response of structures for analysis with near-field input data. For considering directional effects exclusively, all of input accelerations are normalized to have PGA of 0.35 g. It is noteworthy that several methods are available based on seismic standards for earthquake scaling for designing new structures and retrofitting of them. For a specific intensity, the response spectrum of records should be within a specific time period range of standard designing spectrum; or all of the response spectrum of records should be equal with main time period of structures. This method is appropriate for designing of new structures. However, in this study both far-field and near-field records are scaled to a same and constant PGA which is 0.35 g. The reason for this is that the goal of this study is comparing the behavior of models and determining the reliability capability of them and not designing of retrofitting of them. The second reason is that based on ASCE/SEI 41-13 (ASCE, 2017), PGA for both near-field and far-field records for an area with constant earthquake hazard is suggested.

### Modeling and Reliability Capability Analyses

For conducting nonlinear dynamic analysis on numerical models and evaluation of reliability capability, OpenSees program (OpenSees, 2006) is used as a finite element software which has been developed by UC Berkeley since 1990. The program is suitable for both micro and macro analyses using different structural elements, material properties and analytical methods. The reliability capability of steel frames has been investigated based on Monte Carlo simulation for time

variable conditions considering variation of reliability capability of frames for each input seismic records. Structural uncertainty parameters used for this study is shown in Table 3. Based on reliability capability analysis using Monte Carlo method, damage likelihood and reliability capability index of frames for different functional levels are calculated. The deformational threshold in different functional levels with unlimited usage capability, the health safety and the failure threshold are determined based on ASCE/SEI 41-13 (ASCE, 2017) according to the following equation:

$$\delta_i = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \quad (4)$$

### Discussion

Based on assumed structural variables which are used in the simulation, reliability capability index of each frame for different functional levels are calculated. Table 4 shows the reliability capability index and the damage likelihood of the frame with three floors. The results are determined for 100,000 random samples. The same analytical method is used to evaluate the reliability capability index and the damage likelihood for other frames too.

### Results

Tables 4 to 8 show the reliability capability index and damage probability for all cases of this study. The results are provided for all the earthquake records used in this investigation.

**Table 4:** Reliability capability index and damage probability of the 3-floor frame

Earthquake record	First operational method		Second operational method		Third operational method	
	Damage probability	Reliability capability	Damage probability	Reliability capability	Damage probability	Reliability capability
FF1	0.1111	1.2205	0.0850	1.3722	0.0669	1.4987
FF2	0.0499	1.6451	0.0399	1.7513	0.0328	1.8399
FF3	0.0172	2.1151	0.0086	2.3823	0.0052	2.5584
FF4	≈0	>8	≈0	>8	≈0	>8
FF5	≈0	>8	≈0	>8	≈0	>8
FF6	0.0744	1.4432	0.0592	1.6167	0.0395	1.7560
FF7	0.0028	2.7685	0.0014	2.9881	0.0002	3.5010
FF8	≈0	>8	≈0	>8	≈0	>8
FF9	≈0	>8	≈0	>8	≈0	>8
FF10	0.0465	1.6791	0.0342	1.8217	0.0254	1.9523
NF1	0.0224	2.0058	0.0146	2.1797	0.0101	2.3203
NF2	0.0081	2.4017	0.0036	2.6827	0.0010	3.0726
NF3	0.0130	2.2250	0.0089	2.3697	0.0060	2.5076
NF4	0.0056	2.5305	0.0044	2.6170	0.0036	2.6845
NF5	0.0072	2.4440	0.0042	2.6318	0.0028	2.7633
NF6	0.0426	1.7205	0.0326	1.8427	0.0241	1.9747
NF7	0.0135	2.2112	0.0103	2.31424	0.0080	2.4080
NF8	0.0504	1.6404	0.0390	1.7620	0.0309	1.8664
NF9	≈0	>8	≈0	>8	≈0	>8
NF10	0.1603	0.9928	0.1279	1.1361	0.1032	1.2633

**Table 5:** Reliability capability index and damage probability of the 5-floor frame

Earthquake record	First operational method		Second operational method		Third operational method	
	Damage probability	Reliability capability	Damage probability	Reliability capability	Damage probability	Reliability capability
FF1	0.0773	1.4230	0.0620	1.5375	0.0489	1.6555
FF2	0.0530	1.6161	0.0455	1.6891	0.0390	1.7620
FF3	0.0129	2.2275	0.0079	2.3295	0.0076	2.4255
FF4	≈0	>8	≈0	>8	≈0	>8
FF5	≈0	>8	≈0	>8	≈0	>8
FF6	0.0189	2.0763	0.0138	2.2021	0.0102	2.3154
FF7	≈0	>8	≈0	>8	≈0	>8
FF8	≈0	>8	≈0	>8	≈0	>8
FF9	≈0	>8	≈0	>8	≈0	>8
FF10	0.0402	1.7473	0.0329	1.8389	0.0273	1.9209
NF1	0.0159	2.1454	0.0118	2.2606	0.0094	2.3489
NF2	0.0001	3.6420	≈0	>8	≈0	>8
NF3	0.0112	2.2824	0.0088	2.3736	0.0062	2.4956
NF4	0.0052	2.5585	0.0036	2.6866	0.0016	2.9303
NF5	0.0050	2.5690	0.0037	2.6762	0.0022	2.8381
NF6	0.0298	1.8823	0.0240	1.9768	0.0183	2.0881
NF7	0.0154	2.1579	0.0120	2.2546	0.0084	2.3878
NF8	0.0331	1.8363	0.0281	1.9084	0.0239	1.9783
NF9	≈0	>8	≈0	>8	≈0	>8
NF10	0.0849	1.3726	0.0702	1.4736	0.0557	1.5919

**Table 6:** Reliability capability index and damage probability of the 7-floor frame

Earthquake record	First operational method		Second operational method		Third operational method	
	Damage Probability	Reliability capability	Damage probability	Reliability capability	Damage probability	Reliability capability
FF1	0.0488	1.6558	0.0357	1.8027	0.0238	1.9793
FF2	0.0378	1.7761	0.0317	1.8556	0.0255	1.9498
FF3	0.0217	2.0197	0.0165	2.1303	0.0127	2.2349
FF4	≈0	>8	≈0	>8	≈0	>8
FF5	≈0	>8	≈0	>8	≈0	>8

**Table 6:** Continue

FF6	0.0411	1.7386	0.0323	1.8475	0.0249	1.9604
FF7	0.000056	3.8618	≈0	>8	≈0	>8
FF8	≈0	>8	≈0	>8	≈0	>8
FF9	≈0	>8	≈0	>8	≈0	>8
FF10	0.0255	1.9511	0.0187	2.0793	0.0134	2.2122
NF1	0.1075	1.2396	0.0691	1.4824	0.0438	1.7071
NF2	0.0005	3.2848	0.0002	3.5192	0.000058	3.8550
NF3	0.0259	1.9442	0.0226	2.0023	0.0195	2.0635
NF4	0.0051	2.5683	0.0031	2.7357	0.0017	2.9275
NF5	0.0034	2.6982	0.0010	3.0698	0.0000	4.3633
NF6	0.0231	1.9924	0.0162	2.1397	0.0118	2.2635
NF7	0.0002	3.5082	0.0000	3.8267	0.0000	4.4192
NF8	0.0318	1.8546	0.0198	2.0558	0.0120	2.2539
NF9	≈0	>8	≈0	>8	≈0	>8
NF10	0.0697	1.4774	0.0575	1.5760	0.0462	1.6824

**Table 7:** Reliability capability index and damage probability of the 10-floor frame

Earthquake record	First operational method		Second operational method		Third operational method	
	Damage Probability	Reliability capability	Damage probability	Reliability capability	Damage probability	Reliability capability
FF1	0.0203	2.0459	0.0124	2.2437	0.0065	2.4824
FF2	0.0231	1.9923	0.0184	2.0868	0.0143	2.1867
FF3	0.0312	1.8633	0.0259	1.9440	0.0209	2.0336
FF4	≈0	>8	≈0	>8	≈0	>8
FF5	≈0	>8	≈0	>8	≈0	>8
FF6	0.0014	2.9836	0.0006	3.2223	0.0002	3.5252
FF7	≈0	>8	≈0	>8	≈0	>8
FF8	≈0	>8	≈0	>8	≈0	>8
FF9	≈0	>8	≈0	>8	≈0	>8
FF10	0.0443	1.7020	0.0350	1.8118	0.0267	1.9314
NF1	0.0276	1.9162	0.0214	2.0250	0.0170	2.1177
NF2	0.0048	2.5848	0.0029	2.7487	0.0013	2.9955
NF3	0.0099	2.3293	0.0070	2.4544	0.0042	2.6355
NF4	0.0071	2.4511	0.0047	2.5961	0.0034	2.7034
NF5	0.0006	3.2385	≈0	>8	≈0	>8
NF6	0.0279	1.9110	0.0196	2.0619	0.0154	2.1594
NF7	0.0003	3.3774	0.0001	3.6705	0.0000	4.0527
NF8	0.0142	2.1903	0.0079	2.4092	0.0038	2.6623
NF9	≈0	>8	≈0	>8	≈0	>8
NF10	0.0628	1.5311	0.0403	1.7463	0.0199	2.0555

**Table 8:** Reliability capability index and damage probability of the 15-floor frame

Earthquake record	First operational method		Second operational method		Third operational method	
	Damage Probability	Reliability capability	Damage probability	Reliability capability	Damage probability	Reliability capability
FF1	0.0050	2.5728	0.0023	2.8249	0.0011	3.0521
FF2	0.0045	2.6067	0.0029	2.7491	0.0016	2.9318
FF3	0.0565	1.5848	0.0432	1.7136	0.0329	1.8385
FF4	≈0	>8	≈0	>8	≈0	>8
FF5	≈0	>8	≈0	>8	≈0	>8
FF6	0.0008	3.1408	≈0	>8	≈0	>8
FF7	≈0	>8	≈0	>8	≈0	>8
FF8	≈0	>8	≈0	>8	≈0	>8
FF9	≈0	>8	≈0	>8	≈0	>8
FF10	0.0355	1.8054	0.0277	1.9153	0.0219	2.0154
NF1	0.0195	2.0635	0.0147	2.1777	0.0118	2.2621
NF2	0.0045	2.6055	0.0026	2.7841	0.0006	3.2323
NF3	0.0001	3.5490	0.0000	4.0172	0.0000	4.4712
NF4	0.0053	2.5501	0.0038	2.6645	0.0029	2.7524
NF5	0.0006	3.2379	0.0001	3.6773	0.0000	4.1018
NF6	0.0096	2.3400	0.0049	2.5817	0.0002	3.5368
NF7	0.0000	3.8624	≈0	>8	≈0	>8
NF8	0.0314	1.8599	0.0177	2.1032	0.0072	2.4457
NF9	≈0	>8	≈0	>8	≈0	>8
NF10	≈0	>8	≈0	>8	≈0	>8



## Conclusion

In this investigation, reliability capability and damage likelihood of steel moment frames against ten different near-field and far-field earthquake input parameters have been conducted. The models were based on five different frames with 3, 5, 7, 10 and 15 floors with the floor height of 3.2 m and span length of 5 m. According to results, different responses have been recorded for different acceleration input for near-field and far-field zones. As was observed in a specific structure for different accelerograms in the near field and far field different answers obtained. The reliability capability of steel frames for far-field records were more than near-field ones. Therefore, frames facing near-field seismic loads are experience upper values of the functionality limit in longer time intervals. This means that these frames located in near-field areas with high hazard of earthquake will experience severe damages. Determining the reliability capability using Monte Carlo method demonstrated that the response of structures is highly dependent to the distribution of random variables, their means and standard deviations. Based on the results, it was seen that when the standard deviation of parameters is higher, the convergence of analytical responses decreases.

## Acknowledgement

Author appreciate and grateful to the E. Jahani and H. Hamidi Jamnani for their valuable help during 2013-15.

## Ethics

This article is original and contains unpublished material. Author declare that are not ethical issues and no conflict of interest that may arise after the publication of this manuscript.

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