Research Note

Fragility Curve Development for Assessing Midrise Steel Building with Buckling Resistant Braced System Having Vertical Irregularity

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Received: 22/08/2015
Accepted: 16/12/2015

ABSTRACT

In this paper, the fragility curves have been developed for assessing vertically irregular midrise steel building with Buckling Resistant Braced (BRB) system. The effect of different vertical irregularities of mass, stiffness and the concurrent variation of stiffness and strength was investigated in the seismic response of a ten-story steel building. The fragility curves of both the regular and irregular structures were developed through the incremental dynamic analysis and the effects of vertical irregularities were evaluated in the seismic performance of the structure. Fragility curves show that among all the vertical irregularities, variation in the mass has little effect on the probability exceedance of demand from capacity. Meanwhile, the concurrent variation of stiffness and strength shows a significant increase in the probability exceedance of demand from capacity, especially through the nonlinear phase of structural behavior, through the collapse prevention to the global instability limit states.

Keywords:
Fragility curve; Buckling resistance brace; Vertical irregularity; Incremental dynamic analysis

1. Introduction

Irregularity of the buildings is classified into two categories according to the seismic design code of Iran [1]; irregularity in plan and irregularity in height. The variation in the dynamic characteristics of structures (i.e. mass, stiffness and strength) over the building height causes vertical irregularity. These irregularities are known as non-geometric irregularities in some articles [2]. The effect of vertical irregularity on seismic response of structures has been studied by many researchers. Chintanapakdee and Chopra [3] compared the seismic demands of vertically irregular and regular frame buildings by using nonlinear time history analysis. The results showed that a soft and weak story make increase in the drift demand of the irregular floor and some adjacent stories. Tremblay and Poncet [4] focused on the seismic performance of mass irregular steel building with concentrically two dimensional braced frames as lateral resisting system. All the structures have been designed based on static and nonlinear dynamic methods. The results showed that there is no significant difference on the seismic performance of these frames whether they are being designed by static or nonlinear dynamic methods. Pirizadeh and Shakib [2] performed probabilistic seismic performance evaluation on a ten-story building with different irregularity over the height. It was observed that the irregularity due to the variation in dynamic characteristic of structure (e.g. mass, stiffness and strength) makes variation in the structural performance especially at the nonlinear part of structural behavior.

In spite of considering irregularity in structures, choosing an appropriate lateral resisting system for building is another concern to resist earthquake loads. Braced frame is a common seismic resisting system, which is popular in common steel structures of Iran. In comparison to the moment resisting systems, braced frame have better resistance against lateral loads, due to higher lateral stiffness. There are three types of common bracing systems: Concentrically Braced Frame (CBF), Eccentrically Braced Frame (EBF) and Buckling Restrained Braced Frame (BRBF). Through the past decade, great attention has been paid to this lateral resisting system. Türker and Bayraktar [5] performed experimental and numerical investigations on the different bracing configurations in steel buildings. The stiffness properties of different braced frames with various configurations (i.e. X, V, inverse V and K) were evaluated. It was observed that the X bracing system has higher stiffness than the other ones. Mahmoudi and Zaree [6] evaluated the modification factor of the conventional concentric braced frames and buckling restrained braced frames. Static nonlinear analysis was performed on the various bracing configurations (i.e. X, V and inverse V). Results showed that the structures with the BRBFs had a greater response modification factors than the CBFs. Additionally, it was observed that the response modification factors of the BRBFs were higher than the CBFs. Alipour and Aghakouchak [7] evaluated the effects of different parameters on the behavior of the CBFs under cyclic loads. Nonlinear analyses were performed by considering an inelastic finite element model. It was observed that the seismic performance of the CBFs improved by considering the inelastic demands of the gusset plate in corners and at the middle of the brace.

The major problem of the common braced system during strong ground motions is the buckling behavior under compression loads. The BRBF was introduced in 1970s to increase the buckling resistance of the braces and prevent their global buckling behavior [8]. This system has symmetric tension and compression yielding capacity [9]. Studies have been performed to evaluate the overall performance of the BRB system. Asgarian and Shokrgozar [10] performed nonlinear incremental dynamic analysis, linear dynamic analysis and static pushover analysis to investigate the response modification factor of the BRBFs. The effects of the height of the structures and various configurations of bracing systems on the response modification factor were evaluated. Jiang et al. [11] studied the contact force between the steel core and the casing members. They suggested a core thickness, gap and core width-to-thickness ratio by using numerical results. Atlayan and Charney [12] introduced a new type of BRBFs called "hybrid" that consisted of various materials in the core of the brace system. To compare the seismic performance of the new BRBFs and conventional BRBFs, nonlinear static pushover and nonlinear incremental dynamic analyses were performed. Results indicated that the hybrid BRBFs has better seismic performance in comparison to the conventional BRBFs.

Evaluation of the seismic performance of the load resisting systems can be performed by using different methods. Performance Based Earthquake Engineering (PBEE) is an efficient framework that can achieve the overall structure performance. Probabilistic seismic demand analysis is a conceptual part of PBEE that incorporates probability theorem in the analysis procedure [13]. Evaluation of the structural performance in the form of fragility functions gives the vulnerability of the structures under specific ground motion intensity. It is a useful tool that gives the probability of damage exceedance in the structure. Lignos and Karamanci [14] developed fragility curves for the steel bracing systems based on drift evaluation. They considered local and flexural buckling in the final results. They suggested that fragility curves are appropriate for evaluation of the seismic vulnerability of the CBFs. Tsai [15] suggested a performance-based design approach in retrofitting the regular steel braced buildings. By performing nonlinear dynamic analysis, the accuracy of this approach was verified. Ozel and Güneyisi [16] investigated the effectiveness of various configurations of the EBFs on the seismic performance of the retrofitted concrete buildings. According to the given fragility functions, using the EBFs in the concrete buildings improved the seismic performance of this type of building.

The aim of this paper is to develop the fragility curve for assessing midrise steel building with BRBF system having vertical irregularity. The effects of different vertical irregularities of mass, stiffness and the concurrent variation of stiffness and strength were investigated. The results were prepared in the form of fragility curves that represent the probability exceedance of demand from capacity at various seismic intensities.

2. Structural Models

2.1. Reference Regular Building

In this study, a ten-story steel frame building is selected as the reference structure. As shown in Figure (1), the structure has two bays in each direction. The BRBF was selected as the lateral load resisting system. This system consists of various components: steel core, casing, mortar and deboning material. A detailed view of this system has been shown in Figure (2). In this system, steel core provides stiffness against axial loads and flexural stiffness of the casing improve the buckling resistance of the system [11].

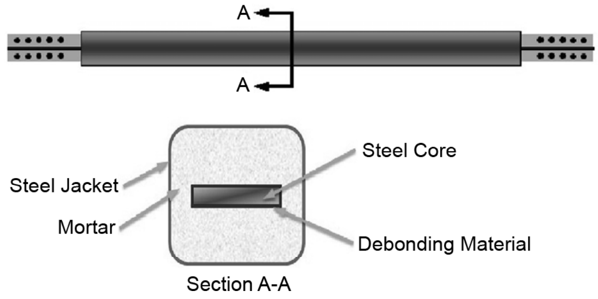

Figure 2. A detailed view of the BRB.

Designing of the frame elements was performed according to the earlier Iranian Seismic Design Code [1]. The site-specific acceleration of 0.35 g (i.e. Tehran city) was assumed in the design procedure. The soil material beneath the building was selected as condense sand. This soil is classified in type II with the shear wave velocity of 360-750 m/s to the depth of 30 m, according to the site classification of Iranian Seismic Design Code [1]. The gravity loads were considered based on the common residential buildings in Iran, so that 0.7 ton/m^2 for dead load and 0.2 ton/m^2 for live load were assigned to each story floor [17]. The fundamental period of the reference structure is $T_1 = 1.2$ s. Table (1) shows the section profile of the frame elements. As shown in Table (1), a uniform distribution of the dynamic

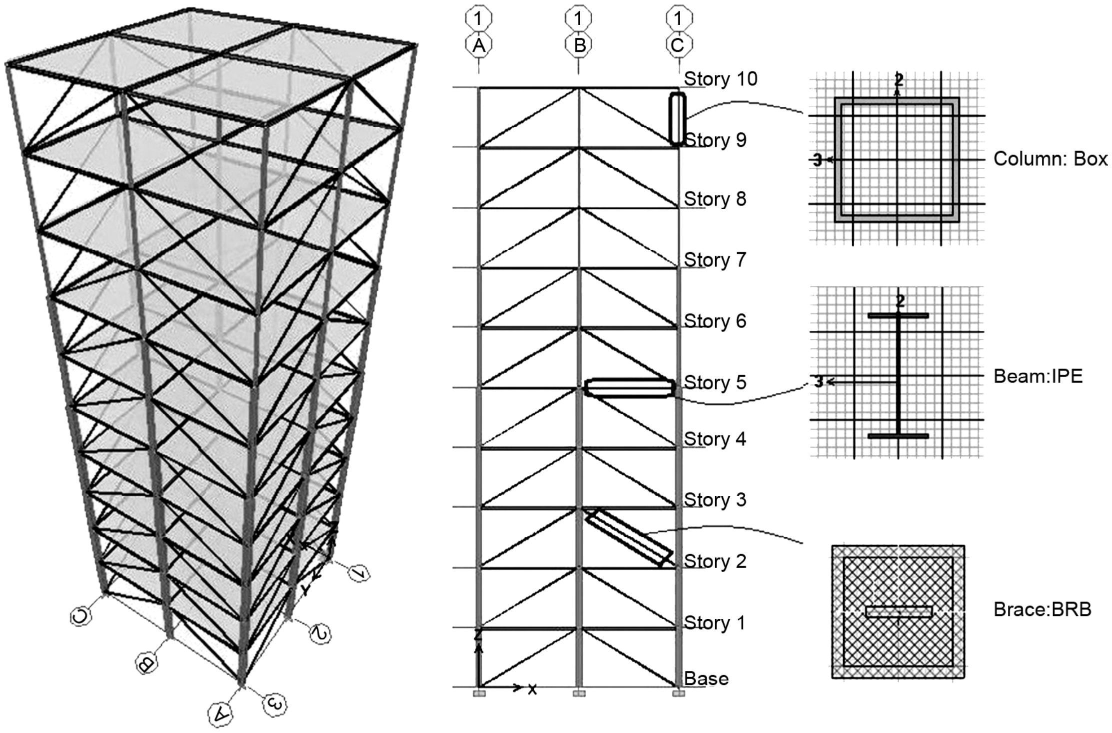

Figure 1. The three-dimensional and elevation view of the reference structure.

characteristic (e.g. mass, stiffness and strength) was considered over the height of the regular frame.

2.2. Vertically Irregular Buildings

In this study, various irregularities were selected by changing the distribution of the dynamic properties of structure (e.g. mass, stiffness and strength) along its height. As shown in Table (2), the modification factors (i.e. factor 2 for mass irregularity and factor 0.6 for other types of irregularity) are used to change the mass, stiffness and strength of irregular floors through the structure height. To compare the seismic performance of the regular and irregular buildings, dynamic characteristics of the structures (e.g. fundamental period, damping and yield base shear) were considered identical by multiplying the uniform scale factors in all stories properties (α, β, λ,

Table 1. The design sections of the frame elements.

Brace (mm^2)*	Beam (mm)	Column (mm)	Story
1600	IPE 100	Box 300x300x20	1-2-3-4-5
900	IPE 100	Box 200x200x15	6-7
700	IPE 100	Box 100x100x15	8-9-10

*Numbers show the area of the steel core of the BRB

Table 2. The Vertically irregular distribution of mass, stiffness and concurrent variation of stiffness and strength through the structure height.

Story	Regular	Irregularity Through the 1st to the 5th Stories	Irregularity in the 1st Story	Irregularity in the 5th Story	
10	m	m	m	λm	Mass Irregularity*
9	m	m	m	λm	
8	m	m	m	λm	
7	m	m	m	λm	
6	m	m	m	λm	
5	m	$2\alpha m$	$2\beta m$	$2\lambda m$	
4	m	$2\alpha m$	$2\beta m$	λm	
3	m	$2\alpha m$	$2\beta m$	λm	
2	m	$2\alpha m$	$2\beta m$	λm	
1	m	$2\alpha m$	$2\beta m$	m	
	Regular	M(2)1:5	M(2)1	M(2)5	
10	k''	Yk''	$\delta k''$	$\Omega k''$	Stiffness Irregularity**
9	k''	Yk''	$\delta k''$	$\Omega k''$	
8	k''	Yk''	$\delta k''$	$\Omega k''$	
7	k'	Yk'	$\delta k'$	$\Omega k'$	
6	k'	Yk'	$\delta k'$	$\Omega k'$	
5	k	$0.6Yk$	δk	$0.6\Omega k$	
4	k	$0.6Yk$	δk	Ωk	
3	k	$0.6Yk$	δk	Ωk	
2	k	$0.6Yk$	δk	Ωk	
1	k	$0.6Yk$	$0.6\delta k$	Ωk	
	Regular	K(0.6)1:5	K(0.6)1	K(0.6)5	
10	k'', s''	$\eta k'', \eta s''$	$\hbar k'', \hbar s''$	$\zeta k'', \zeta s''$	Combination of Stiffness & Strength Irregularity***
9	k'', s''	$\eta k'', \eta s''$	$\hbar k'', \hbar s''$	$\zeta k'', \zeta s''$	
8	k'', s''	$\eta k'', \eta s''$	$\hbar k'', \hbar s''$	$\zeta k'', \zeta s''$	
7	k', s'	$\eta k', \eta s'$	$\hbar k', \hbar s'$	$\zeta k', \zeta s'$	
6	k', s'	$\eta k', \eta s'$	$\hbar k', \hbar s'$	$\zeta k', \zeta s'$	
5	k, s	$0.6\eta k, 0.6\eta$	$\hbar k, \hbar s$	$0.6\zeta k, 0.6\zeta s$	
4	k, s	$0.6\eta k, 0.6\eta$	$\hbar k, \hbar s$	$\zeta k, \zeta s$	
3	k, s	$0.6\eta k, 0.6\eta$	$\hbar k, \hbar s$	$\zeta k, \zeta s$	
2	k, s	$0.6\eta k, 0.6\eta$	$\hbar k, \hbar s$	$\zeta k, \zeta s$	
1	k, s	$0.6\eta k, 0.6\eta$	$0.6\hbar k, 0.6\hbar$	$\zeta k, \zeta s$	
	Regular	K,S(0.6)1:5	K,S(0.6)1	K,S(0.6)5	

* m Refers to Story Mass

** k, k' and k'' Refer to Story Stiffness ($k > k' > k''$)

*** s, s' and s'' Refer to Story Strength ($s > s' > s''$).

γ, ∂, Ω, η, $\hbar$ and ζ are the modification coefficients in Table (2)). Similar to the previous study [2], irregularities were located in three different positions: the first story (1), the fifth story (5) and through the 1st to the 5th floors (1:5).

2.3. Numerical Model and Material Behavior

The three-dimensional model of the structures were built in PERFORM-3D structural analysis software [18] for nonlinear analyses. The P-M-M model of hinge was used to consider the interaction effects of axial loading and bending strength in the column elements. The backbone cure of moment-rotation of the column elements is presented in Figure (3). To consider the nonlinear behavior of the BRB elements, an axial hinge is considered in each brace element. The backbone curve of the BRB is shown in Figure (4). In the BRB backbone curve, ω and β are the strength adjusting parameters and:

$$R_y = F_{ye} / F_{yc} \tag{1}$$

where F_c is the brace core yield stress and F_{ye} is the expected yield stress:

$$F_{ye} = 1.25 F_y \tag{2}$$

In this study, R_y, ω and β were assumed 1.1, 1.25 and 1.1, respectively [19]. K_0 is the initial stiffness parameter that is expressed as:

$$A_s E / L \tag{3}$$

where, L is 70% of the actual center to center length of the brace element and A_s is the area of steel core

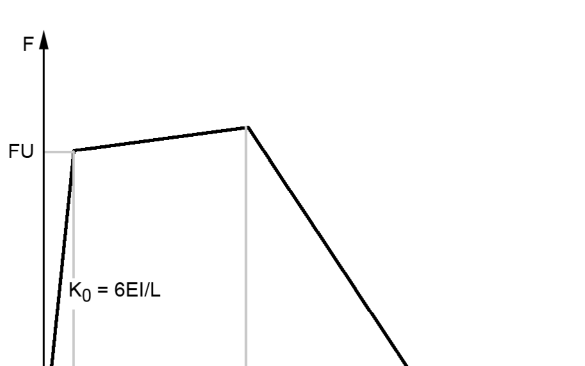

Figure 3. The backbone curve of the moment-rotation of column elements.

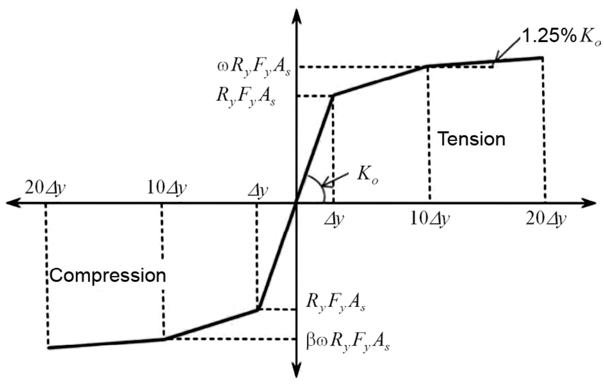

Figure 4. The backbone curve of the BRB elements.

and E is the modulus of elasticity of steel material [19].

3. Methodology

3.1. Incremental Dynamic Analysis

The Incremental Dynamic Analysis (IDA) is an appropriate method that uses nonlinear dynamic analysis to determine a complete range of structural response from elastic phase to the global instability. In this method, the earthquake ground motions are scaled to different levels of intensity [20-21]. Summarizing the structural response relative to the ground motion intensity gives the IDA curve that is being used to specify the structural performance levels.

The IDA curve represents the structural response for a given level of earthquake intensity. The Intensity Measure (IM) is a parameter that represents the seismic hazard of ground motions at the building site. To predict the degree of damages to the structures, the Engineering Damage Parameter (EDP) is introduced. It gives the structural response for a specific level of ground motion intensity. In this study, the seismic performance of the regular and irregular buildings was determined by using the IDA method. The spectral acceleration of the 1st period of the structures was selected as the IM index and the maximum inter-story drift ratio was chosen as the EDP. As shown in Table (3), twenty earthquake records that has been recorded on soil type II with magnitude of 6 to 7.3 were used in the dynamic analyses. Each record has been scaled to various levels of intensity. The structural response under the scaled earthquake ground motions was evaluated.

Table 3. Ensemble of earthquake records in the IDA analysis.

No.	Event	Magnitude	R (km)	PGA (g)
1	Northridge, 1994 (Leona Valley)	M(6.7)	37.8	0.106
2	Loma Prieta, 1989 (Fremont-Mission)	M(6.9)	43.1	0.124
3	Northridge, 1994 (Mabilu)	M(6.7)	35.2	0.118
4	Northridge, 1994 (New port Bch)	M(6.7)	32.3	0.197
5	Landers, 1992 (Riverside Airport)	M(7.3)	96.4	0.043
6	N. Palm Springs, 1986 (Riverside Airport)	M(6)	71.7	0.04
7	Northridge, 1994 (Lake Hughes)	M(6.7)	32.3	0.063
8	Northridge, 1994 (Sanberg – Bald Mtn)	M(6.7)	43.4	0.98
9	San Fernando, 1971 (Lake Hughes)	M(6.6)	20.3	0.283
10	Northridge, 1994 (Burbank)	M(6.7)	20	0.163
11	Northridge, 1994 (Pacific Palisades)	M(6.7)	32.3	0.197
12	Northridge, 1994 (Baldwin Hills)	M(6.7)	31.3	0.168
13	Northridge, 1994 (Mt Wilson)	M(6.7)	36.1	0.234
14	San Fernando, 1971 (Castaic)	M(6.6)	24.2	0.268
15	N. Palm Springs, 1986 (Hurkey Creek)	M(6)	34.9	0.24
16	Northridge, 1994 (Antelope Buttes)	M(6.7)	47.3	0.068
17	Duzce, Turkey, 1999 (Sakarya)	M(7.1)	49.9	0.016
18	Northridge, 1994 (Duarte)	M(6.7)	51.6	0.079
19	Northridge, 1994 (Riverside Airport)	M(6.7)	101.6	0.064
20	San Fernando, 1971 (Pearblossom)	M(6.6)	38.9	0.136

3.2. Structural Performance Levels

For an ensemble of EDP for a given IM, the performance objectives of each structure were given from the median curve of IDA. Four common performance levels were assumed based on ASCE 41-13 [22], FEMA356 [23], FEMA351 [24], FEMA450 [25].

The Immediate Occupancy (IO) is defined as the elastic slope of the median IDA curve reaches to the inelastic region, since it is expected that the building could be returned to its serviceability without major repairs [26]. The Collapse Prevention (CP) is expressed as the local tangent of the IDA curve reaches 20% of the elastic slope while the maximum drift demand is less than 0.1. The Life Safety (LS) performance level may do so by interpolating between the acceptance criteria provided for the Collapse Prevention and Immediate Occupancy levels. The Global Instability (GI) is happened when the flat-line is reached [20]. Seismic capacity of the structures at these performance objectives are presented in Table (4).

3.3. Fragility Curves

Fragility functions are appropriate to evaluate the seismic vulnerability of structures against the hazard of earthquakes. In this study, the fragility functions were used to assess the effects of different irregularities on the seismic performance of BRBF structures. In this procedure, the probability exceedance of demand from capacity of different

Table 4. Seismic capacity of the structures at the performance objectives.

		IO		LS		CP		GI	
		S_a	θ	S_a	θ	S_a	θ	S_a	θ
Regular		0.48g	0.02	0.95g	0.033	1.15g	0.046	1.2g	0.055
Mass Irreg.	M(2)1:5	0.46g	0.02	0.95g	0.032	1.1g	0.044	1.2g	0.059
	M(2)1	0.47g	0.02	0.95g	0.033	1.15g	0.046	1.2g	0.054
	M(2)5	0.46g	0.02	0.95g	0.033	1.14g	0.046	1.2g	0.057
Stiffness Irreg.	K(0.6)1:5	0.68g	0.02	0.98g	0.028	1.2g	0.035	1.2g	0.037
	K(0.6)1	0.6g	0.02	0.96g	0.033	1.31g	0.047	1.4g	0.059
	K(0.6)5	0.52g	0.02	0.94g	0.031	1.1g	0.041	1.2g	0.061
Stiffness & Strength Irreg.	K&S(0.6)1:5	0.63g	0.015	0.67g	0.016	0.7g	0.018	0.71g	0.021
	K&S(0.6)1	0.79g	0.019	0.85g	0.022	0.89g	0.025	0.9g	0.028
	K&S(0.6)5	0.69g	0.016	0.7g	0.017	0.7g	0.017	0.71g	0.021

levels of ground motion intensities specifies for different structural limit states. Based on the median IDA curve, the relation between the *EDP* at a specific *IM* level is expressed as:

$$\ln EDP(IM) = \alpha + \beta \ln (IM) \quad (4)$$

where, α and β are the regression coefficients. It was observed that the distribution of the EDP for a given *IM* has Log-normal distribution [27], thus:

$$\ln EDP| \; IM = \ln EDP(IM) + \ln \varepsilon (IM) \quad (5)$$

where $\varepsilon (IM)$ is the dispersion of the *EDP* at the specific *IM*. The value of the dispersion is assumed constant for any value of *IM*. Therefore, the standard deviation is obtained as [28]:

$$\sigma_{\ln \varepsilon} = \sqrt{\frac{\sum_{j}^{n} (\ln EDP_j - \alpha - \beta \ln IM_j)^2}{n-2}} \quad (6)$$

In the above equation, *n* represents the number of EDP-IM data pairs while EDP_j and IM_j are values of the j^{th} data pair. For the specific *IM*, the probability of EDP exceeding the damage level L_s at a given $IM(P(EDP \geq L_s \mid IM)$ is obtained from the following equation:

$$(P(EDP \geq L_s \mid IM) = 1 - \int_0^{L_s} \frac{1}{\sqrt{2\pi}\sigma_{\ln\varepsilon} y} \times \exp(-\frac{\ln y - \ln EPD(IM))^2}{2\sigma_{\ln\varepsilon}^2}) dy \quad (7)$$

4. Results and Discussion

To evaluate the effects of vertical irregularities on the seismic fragility of the building, the seismic fragility curves of each irregular building were compared to that of the regular one. Four different spectral accelerations were selected to compare the probability of exceedance of demand from the capacity in different cases of irregularities. Since the main specifications of both the regular and irregular structures were kept identical, the spectral accelerations corresponded to the performance levels of regular structure were chosen in the comparison procedure. Therefore, the probability of exceedance of demand from the capacity of irregular frames were compared with the regular structure at $S_a(T_1, 5\%) = 0.48$ g, 0.95 g, 1.15 g and 1.2 g. These values represent the IM index of regular frame for the IO, LS, CP and GI performance levels, respectively.

4.1. Probability Exceedance of Demand from Capacity at the IO Limit State

For the IO limit state, the probability exceedance of demand from the capacity of different irregular buildings has been compared to the regular frame in $S_a(T_1, 5\%) = 0.48$ g. As shown in Figures (5) to (7), it is observed that different mass irregularities have no significant effect on the probability exceedance of demand from the capacity. In spite of 6% decrease in cases of mass irregularity at the 5th floor and through the 1st to the 5th stories, the increase of the mass at the 1st floor seems to have no effect on the probability exceedance variation.

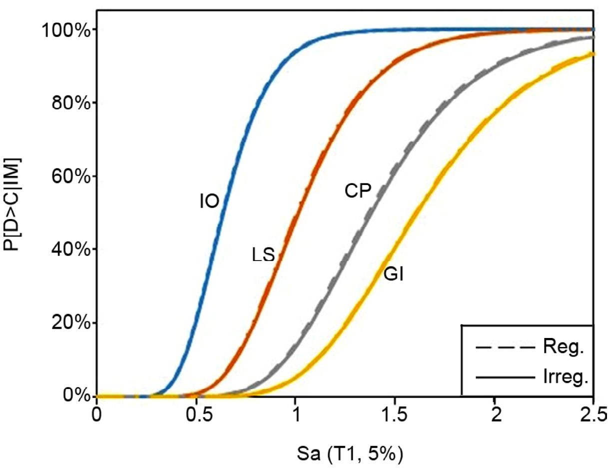

Figure 5. The fragility curves for the BRBF structure with mass irregularity at the 1st floor.

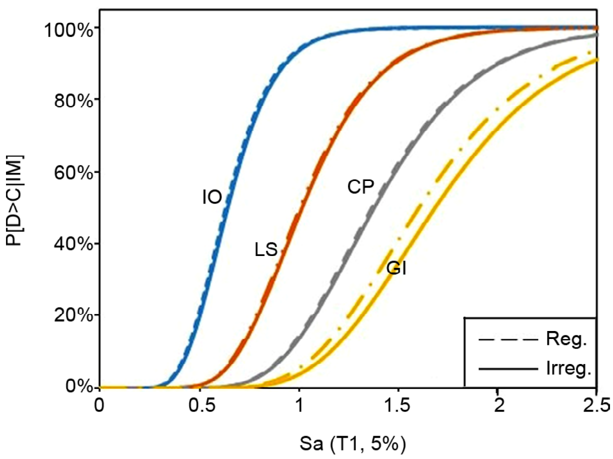

Figure 6. The fragility curves for the BRBF structure with mass irregularity at the 5th floor.

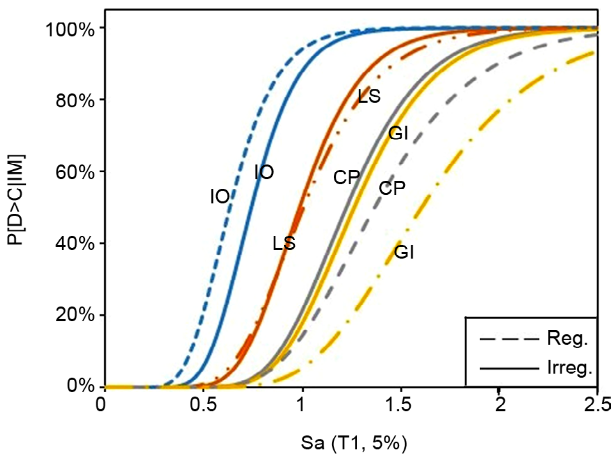

Figure 7. The fragility curves for the BRBF structure with stiffness irregularity through the 1st to the 5th stories.

As Figures (8) and (9) show, in case of stiffness irregularity, it is observed that the probability exceedance of demand from the capacity decreases 18% in case of stiffness irregularity at the 1st and the 5th floor. On the other hand, for stiffness irregularity through the 1st to the 5th stories, Figure (7), this variation decreases up to 76%.

In case of concurrent variation of stiffness and strength, the probability exceedance of demand from the capacity decreases in comparison to the regular structure. Figure (10) shows that in case of irregularity at the 1st floor, the maximum value of probability exceedance of demand from the capacity decreases. In this case, the difference in the probability exceedance of demand from capacity is limited to 94%, in comparison to the regular structure. In other cases, the variation reduces so

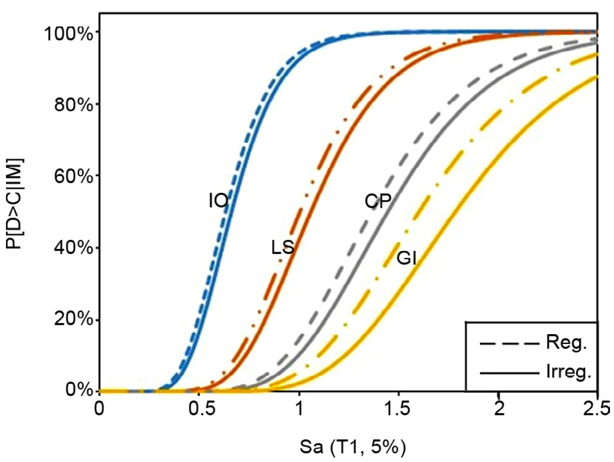

Figure 8. The fragility curves for the BRBF structure with stiffness irregularity at the 1st floor.

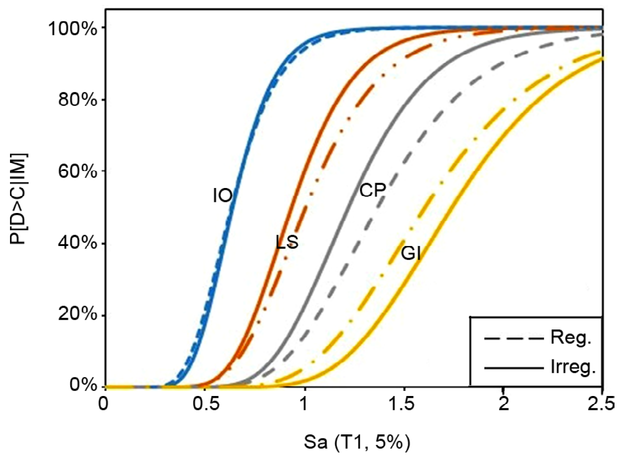

Figure 9. The fragility curves for the BRBF structure with stiffness irregularity at the 5th floor.

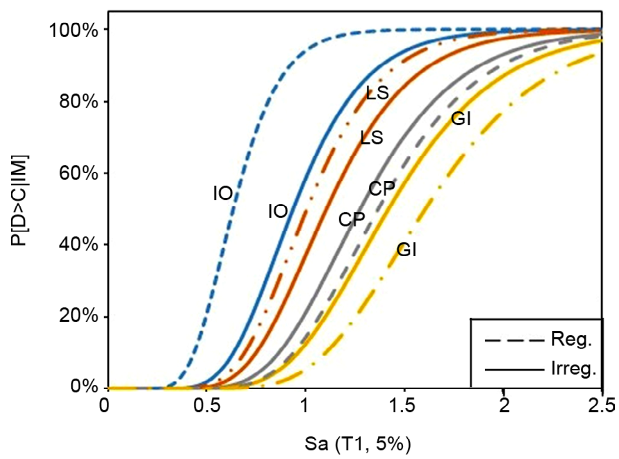

Figure 10. The fragility curves for the BRBF structure with stiffness and strength irregularity at the 1st floor.

that for irregular structure with irregularity at the 5th floor and through the 1st to the 5th stories, the decrease of the probability exceedance of demand from the capacity limited to 59% and 53%, respectively, Figures (10) and (12).

4.2. Probability of Exceedance of Demand from Capacity at LS Limit State

The probability exceedance of demand from capacity of different irregular structures are compared to the regular one at the *IM* index of $S_a(T_1, 5\%) = 0.95$ g for the LS limit state. The results show no significant difference in case of mass irregularity compared to the regular frame. Specifically, Figures (6) and (7) show that mass irregularity at the 1st and the 5th floors causes 2% decrease in the probability exceedance of demand from the capacity, relative to the regular structure. As shown

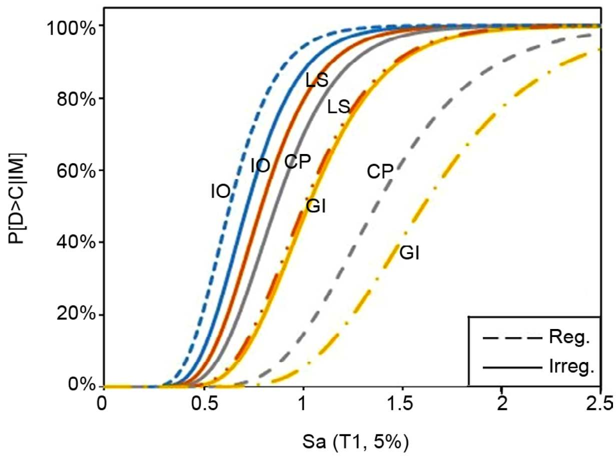

Figure 11. The fragility curves for the BRBF structure with stiffness and strength irregularity through the 1st to the 5th stories.

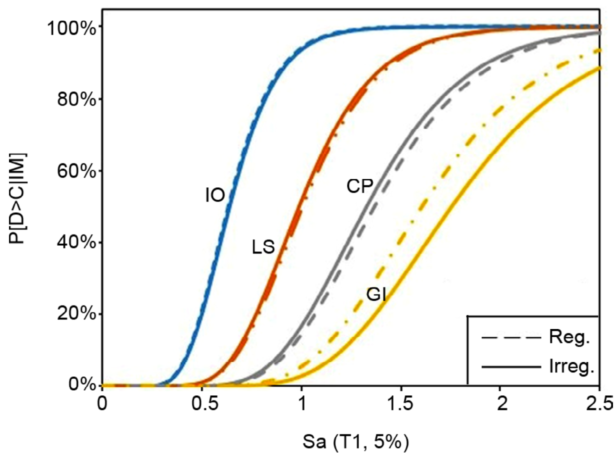

Figure 13. The fragility curves for the BRBF structure with mass irregularity through the 1st to the 5th stories.

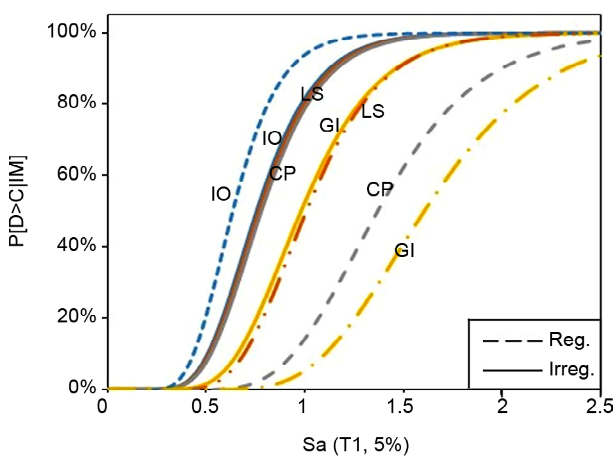

Figure 12. The fragility curves for the BRBF structure with stiffness and strength irregularity at the 5th floor.

in Figure (13), it is observed that in case of mass irregularity through the 1st to the 5th stories, this difference increases to 7%.

As shown in Figure (8), stiffness irregularity at the 1st floor reduces the probability exceedance of demand from the capacity. Meanwhile, in other case of stiffness irregularity, this variation increases up to 26% and 5% for stiffness irregularity at the 5th floor and through the 1st to the 5th stories, respectively, Figures (8) and (11).

Concurrent variation of stiffness and strength through the structure height shows that the probability exceedance of demand from the capacity reduces up to 26% when the irregularity concentrated at the 1st floor, Figure (10). On the other hand as Figure (12) shows, these variations increase 79% in case of irregularity at the 5th floor. Similar result is observed in the other case, Figure (11), so that the probability exceedance of demand from the capacity increases about 76%.

4.3. Probability of Exceedance of Demand from Capacity at CP Limit State

Comparing the probability exceedance of demand from the capacity at the CP performance level is performed in $S_a(T_1, 5\%) = 1.15$ g. As shown in Figures (5) and (6), it is observed that in case of mass irregularity at the 1st and 5th floor, the variation of the probability exceedance of demand from the capacity is about 4%. Meanwhile, Figure (5) shows that 11% increase is observed in case of mass irregularity through the 1st to the 5th stories.

The stiffness irregularity at the 1st floor represents 21% decrease of the probability exceedance of demand from the capacity, in comparison to the regular frame, Figure (8). However, from Figures (8) and (9), 46% and 43% increase is observed for stiffness irregularity at the 5th floor and through the 1st to the 5th stories, respectively.

Figures (10) to (12) show that there is a significant increase in the probability exceedance of demand from the capacity, in case of concurrent variation of stiffness and strength through the structure height. It is observed that in case of irregularity at the 1st floor, the probability exceedance of demand from the capacity increases 32%. In other cases, this variation is more predominant so that for irregularity at the 5th floor and through the 1st to the 5th stories, 218% and 200% increase is obtained, respectively.

4.4. Probability of Exceedance of Demand from Capacity at GI Limit State

To compare various irregularities at the GI performance level, the probability exceedance of demand from the capacity has been compared to the regular frame in $S_a(T_1, 5\%) = 1.2$ g. It is observed that no significant difference appears in the probability exceedance of demand from the capacity for mass irregularity at the 1st floor, Figure (5). On the other hand, Figure (6) shows that mass irregularity of the 5th floor caused 25% decrease in the probability exceedance of demand from the capacity, compared to the regular structure. As shown in Figure (5), in case of mass irregularity through the 1st to the 5th stories the decrease limited to 38%.

Stiffness irregularity at the 1st floor shows 44% reduction, Figure (8). Figure (9) shows that this variation is about 50% in case of irregularity at the 5th floor. However, as shown in Figure (7), stiffness irregularity through the 1st to the 5th stories causes 163% increase in the probability exceedance of demand from the capacity, relative to the regular frame.

In case of concurrent irregularity of stiffness and strength, it is observed that the variation of the probability exceedance of demand from the capacity is more obvious in case of irregularity at the 5th floor with 363% increase, Figure (12). Similarly, Figure (11) shows that in case of irregularity through the 1st to the 5th stories, the increase of the probability exceedance of demand from capacity limited to 344% in comparison to the regular frame. In return, this variation is moderate in case of irregularity at the 1st floor. As shown in Figure (10), in this case, the probability exceedance of demand from the capacity increases up to 81%.

5. Conclusion

In this study, the effects of different types of vertical irregularities on the seismic performance of BRBFs have been investigated. Nonlinear time history analysis was performed on ten-story buildings with two bays in both orthogonal directions. The fragility curves and the probability exceedance of demand from the capacity at the seismic performance levels were obtained through the IDA analysis. According to the obtained results, the following conclusions were made:

The results show that mass irregularity through the structure height has little effect on the probability exceedance of demand from the capacity, in comparison to the other cases of irregularity. In this case of irregularity, it is observed that as the structure has linear behavior (through the IO and LS limit states) no significant difference appear in the probability exceedance of demand from the capacity in comparison to the regular frame. However, the increase of the ground motion intensity causes the structure behave nonlinearly so that through the CP to the GI performance level, the variation of the probability exceedance of demand from the capacity increases up to 38% in comparison to the regular frame.

Results show that the stiffness irregularity causes different effect on the seismic behavior of the irregular structures in comparison with regular structure. The probability exceedance of demand from the capacity decreases, typically at the IO limit state. On the other hand, the stiffness reduction causes an increase in the CP and GI states in comparison with the regular structure.

The concurrent variation of stiffness and strength cause significant increase on the probability exceedance of demand from the capacity at the CP and GI state, mostly. The concurrent irregularity of stiffness and strength at the 5th floor shows 363% increase in the probability exceedance of demand from the capacity in comparison with the regular building. Results show the better performance at the IO level when the irregularity is located at the 1st floor.

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