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A Comparative Study on the Behavior of Steel Moment-Resisting Frames with Different Bracing Systems Based on a Response-Based Damage Index

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Abstract

Seismic rehabilitation of existing buildings is one of the most effective ways to reduce damages under destructive earthquakes. The use of bracings is one of techniques for seismic rehabilitation of steel structures. In this study we aimed to investigate the seismic performance of three 5, 10 and 15-storey steel structures with moment-resisting frames designed three dimensionally in ETABS 2015 application based on first edition of Iranian Standard 2800. Their damage under five ground motions was evaluated using response-based damage model proposed by Ghobara et al. (1999). Then, the structures were rehabilitated with different bracing systems (X, eccentric and concentric V and inverted-V) and, again, their damage under five earthquakes were evaluated and compared with those of moment resisting frames. The pushover analysis results indicated that X-braced frame was the least ductile system but had highest initial stiffness and yield stress. In low-rise building, X-braced frames showed better performance among studied bracing systems compared to moment resisting frames, while mid and high-rise buildings with eccentrically braced frame (EBF) showed the best behavior against earthquakes with the least damage. Moreover, it was found out that EBFs' performance increases by increasing storey height, but for concentrically braced frames (CBFs) it was decreased. We concluded that the use of response-based damage models can be a suitable procedure for estimating the vulnerability of steel structures rehabilitated with bracing system.

Keywords: Steel Moment Resisting Frame; Rehabilitation; Braced Frames; EBF; CBF; Response-Based Damage Model.

1. Introduction

Earthquakes often cause a certain state of damage to structures, the extent of which is generally shown by using a damage index. Various damage indices have been proposed using different parameters such as drift, natural period of structure, energy absorption and cyclic fatigue such as Bozorgnia-Bertero [1], Park-Ang [2], Krawinkler-Zohrei [3], Roufaiel-Meyer [4], Dipasquale-Cakmak [5], and Ghobara [6] indices. They are divided into two broad categories: non-cumulative and cumulative damage indices. "Non-cumulative indices do not include the effects of cyclic loading and often do not reflect the state of damage accurately, whilst the cumulative indices are rather more rational" [7]. An ideal damage index should be between 0 to 1 where 0 refers to the state of elastic response, and 1 indicates the collapse state. Several damage models exist for characterization of structural failure in terms of damage index including strength-based and response-based models. Strength-based damage models were first proposed by Shiga et al. [8] and later used by Yang and Yang [9]. These models depend on the geometry of structural elements such as the column and wall area and their general material properties. The response-based damage indices are divided into three groups: maximum

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deformation (e.g. ductility ratio, interstorey drift, and slope ratio), cumulative damage (e.g. low cycle fatigue), and maximum deformation and cumulative damage (e.g. Park-Ang index) [6].

In literature, there are various works on application of damage indices in the seismic assessment of steel buildings as well as presenting different techniques for damage evaluation. For example, among recent works, Sary et al. [10] used the damage index proposed by [2] in order to find correlation between ground motion parameters and damage. In another study, Estekanchi and Arjomandi [11] investigated different damage indexes based on deformation, energy, modal parameters and low cycle fatigue behavior to find a relationship between their numerical values by employing them in the nonlinear analysis of different low-rise steel structures. According to their results, the Bozorgnia-Bertero, Park-Ang and Krawinkler-Zohrei damage models showed a relatively better association with maximum drift index. Bojórquez et al. [12], by complementing experimental and analytical tests results, introduced an energy-based damage index for multi-degree-of-freedom steel buildings that explicitly causes the effects of cumulative plastic deformation demands. Sinha and Shiradhonkar [13] evaluated the relationship between analytical damage indices and observational damage states and found out that the damage indices cannot provide accurate measurement of the intermediate damage states. Gerami et al. [14] studied seismic vulnerability of irregular steel buildings considering effects of the panel zone, and applying cumulative damage indices. They observed severe damage and collapse due to the dissipation of energy in the initial storeys of low-rise buildings and in the middle storeys of high-rise buildings.

Kamaris et al [15] presented a new damage index for plane steel frames by considering the interaction between axial force and bending moment. The damage index was defined by assuming a linear variation of damage between the two curves; “the first curve was the limit between elastic and inelastic material behavior (damage index = 0), and the second was the limit between inelastic behavior and complete failure (damage index = 1)”. Diaz et al. [16] proposed another new damage index based on nonlinear static analysis and Park-Ang damage index. Their results showed good agreement with those calculated by means of dynamic analyses. Eraky et al. [17] used damage index method for determining local damages in flexural structural elements. Their technique was based on the comparison of modal strain energies at different degradation stages. They showed that the experimental results were in good agreement with numerical results in identifying damages of flexural structural elements. Mirzaaghabeik and Vosoughifar [18] developed two new procedures for measuring the damage index of lightweight steel structures based on a qualitative approach. For this purpose, non-linear static analysis was used. Maleki-Amin and Estekanchi [19] estimated various damage indices for different steel moment frames by applying the endurance time method and concluded that damage estimation by this method with initial target time is not always acceptable. Tan et al. [20] presented a vibration-based technique for prediction of damage, its location and severity in steel beams. For single damage scenarios, the modal strain energy-based damage index and for multiple damage scenarios, artificial neural network were employed. Their results confirmed the feasibility of the proposed method and its application in preventing structural failure. Diaz et al. [21] applied the Park-Ang and the capacity-based damage indices to steel frame structures and conducted deterministic and probabilistic nonlinear static and incremental dynamic analyses to assess seismic performance of low-, mid- and high-rise buildings. In both damage indices, stiffness degradation and energy dissipation were taken into account. Results showed that the lowest contribution of energy dissipation was for the low-rise building. Also, the energy contribution raised with the ductility of the building and with the duration of the strong ground motion.

Steel structures are widespread. They show ductile behavior when are subjected to lateral loading. The use of bracings for seismic rehabilitation of structures provides lower stiffness and resistance for a structure compared to shear walls but such system has lower weight and more useful for architectural purposes [22]. In these systems, the distance between the brace and the frame node is called “link” which provides plastic deformation capacity and dissipate the energy released by the earthquake. Eccentrically Braced Frames (EBFs) and Concentrically Braced Frames (CBFs) are two common bracing systems used in steel structures in addition to moment resisting frames (MRFs) which are allowed in ANSI/AISC 341-10. In EBF, the longitudinal axis of each brace is eccentric to the midpoint of the beam equal to one-half the link length, while in CBF, the longitudinal axis of each brace is concentric to the midpoint of the beam and intersects it [23-25] (see Figure 1). CBFs are used in different forms such as cross, diametric and chevron, while EBFs can be used in different forms depending on the location of the link. EBFs show lateral stiffness similar to that of CBF, and ductility similar to that of the moment frame [26]. EBFs are desirable seismic load resisting systems, since they combine the high elastic stiffness of CBFs with the ductility and stable energy dissipation of MRFs [27]. EBFs have a lower stiffness than CBFs, but show more ductile behavior [22]. Many studies have conducted on these braced frames. For example, Mahmoudi and Zaree [28] evaluated the overstrength of CBFs with different configurations (chevron V, inverted V and X-bracing) considering members post-buckling strength. They conducted pushover analysis and found out that the number of bracing bays and storey height has a little effect on reserve strength due to the post-buckling of brace. Musmar [23] investigated the effect of link on EBF configuration and compared its structural response with those of CBF and MRF. According to his results, it was found that EBF systems with shorter links showed more stiffness. Also, their results were consistent with the results of Berman et al [27] and suggested that EBFs are “efficient laterally stiff framing systems with significant energy dissipation capability”. Using both non-linear static and dynamic analysis, Chimeh and Homami [22] studied the behavior of rehabilitated structures by X braced frames; chevron braced frames

(Inverted-V braced frames and V braced frames), Zipper columns and EBF with long and short link beams. The Zipper columns bracing system and short linked EBF were reported as the most ductile systems while the EBF system had the most efficiency. In another study, the behavior of three different eccentric braces (V, Inverted-V and Diagonal) in 4 and 8-storey structures was investigated by Tande and Sankpal [29]. The frames were assessed by nonlinear static pushover analysis. They observed that the plastic hinges occurred first at the fuse section of braces and later at the compressive members of the eccentric braces. Shen et al. [30] conducted an analytical study on seismic performance of CBFs using two-story X-braced frames with strong and weak braced-intersected beams and inverted V braced frames with and without brace buckling. They found out that the braces in CBFs are fractured often before 2% story drift ratio response mainly because of premature yielding of the beams. Salmasi and Sheidaii [31] assessed strength of EBFs against progressive collapse by using nonlinear static alternate path method. Three different types of bracing, including eccentric inverted V bracing (chevron bracing), eccentric V bracing, and eccentric X bracing were employed. Their results pointed out that dual steel moment frames rehabilitated with EBFs generally show desirable strength against progressive collapse.

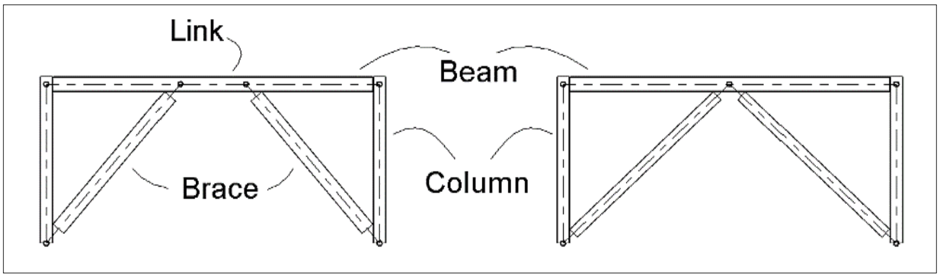


Figure 1. EBF (left) and CBF (right) [24]

Due to the analytical difficulties in assessment of the change in period as a measure of damage, a new approach was proposed by Ghobara et al. in 1999 [6] for determining the change in stiffness of the structure. Among recent studies, no research was found that have employed this damage index model for evaluating the behavior of concentrically and eccentrically braced steel frames. Considering this limitation and due to the importance of steel structures' vulnerability against earthquakes and their rehabilitation, in this paper we attempted to evaluate seismic performance of three lows-, mid- and high-rise steel buildings with moment resistant frames rehabilitated with different bracing systems (X, eccentric and concentric V and Inverted-V) using response-based damage index of Ghobara et al. For this purpose, non-linear static analysis (pushover) was conducted in SAP 2000 application.

2. Methodology

2.1. Modeling Steel Structures

To evaluate performance of steel buildings rehabilitated with bracing systems, three 5-, 10-, and 15-storey structures with moment-resisting frames that were irregular in plan were designed according to the first edition of Standard 2800 [32] and the tenth code of the national building regulations, and three-dimensionally modeled in ETABS 2015 application. They had same framing plan with 4-m span in X and Y directions and floor heights of 15, 30, and 45 m (Figure 2) located on a site with soil type C and high relative risk. Due to higher height, Spectral analysis was used to design the 10, and 15-storey structures. Dimensions of the beams and columns for the three study structures are presented in Table 1. The structures consisted of steel box-shaped columns and steel wide flange I-shaped beam sections.It should be mentioned that the column and beam sections used to design 10 and 15-storey structures are not symmetric. For the roof, concrete blocks were applied supported by steel joists. The properties of the used steel and concrete are presented in Table 2.

The gravity loading were based on national regulations for design load where storey live load, storey dead load and roof live load were obtained as 300, 700 and 150 kg/m^2, respectively. Also, the lateral loading was in accordance with the first edition of Standard 2800. Their importance factor (I), design base acceleration (A), the behavior factor (R) were set as 1, 0.35 and 6 respectively. Furthermore, the reflection coefficient (B) for 5-, 10-, and 15-storey structures was considered as 1.75, 1.24 and 1.01 respectively based on Standard 2800. In this regard, seismic coefficients (C) for these structures were obtained as 0.102, 0.072 and 0.059 respectively. Figure 3 shows 3D model of the structures with moment-resisting frames.

Table 1.Specifications of beams and columns used in the study buildings

Structure	Columns (cm)	Beam (cm)
5-storey	BOX-25×25×0.15 BOX-20×20×0.2	I-27×0.2×13.5×0.2 I-22×0.2×11×0.2 I-20×0.1×10×0.1
10-storey	BOX-30×30×0.2 BOX-30×30×0.15 BOX-25×25×0.15 BOX-20×20×0.2 BOX-15×15×0.2	I-20×0.1×10×0.1 I-20×0.2×10×0.2 I-24×0.2×12×0.2 I-27×0.2×13.5×0.2 I-30×0.2×15×0.2
15-storey	BOX-35×35×0.3 BOX-30×30×0.2 BOX-30×30×0.15 BOX-25×25×0.2 BOX-20×20×0.2	I-20×0.1×10×0.1 I-22×0.2×11×0.2 I-24×0.2×12×0.2 I-27×0.2×13.5×0.2

Table 2.Properties of the steel and concrete materials

Material properties	**Steel**	Grade = ST37 Minimum yield stress= 2400 kgf/cm^2 Minimum tensile stress= 3700 kgf/cm^2 Modulus of elasticity= 2.1×10^6 Kgf/cm^2 Poisson's ratio= 0.3 Density = 7850 Kgf/m^2
	Concrete	Grade = C21 Modulus of elasticity = 2.1×10^5 Kgf/cm^2

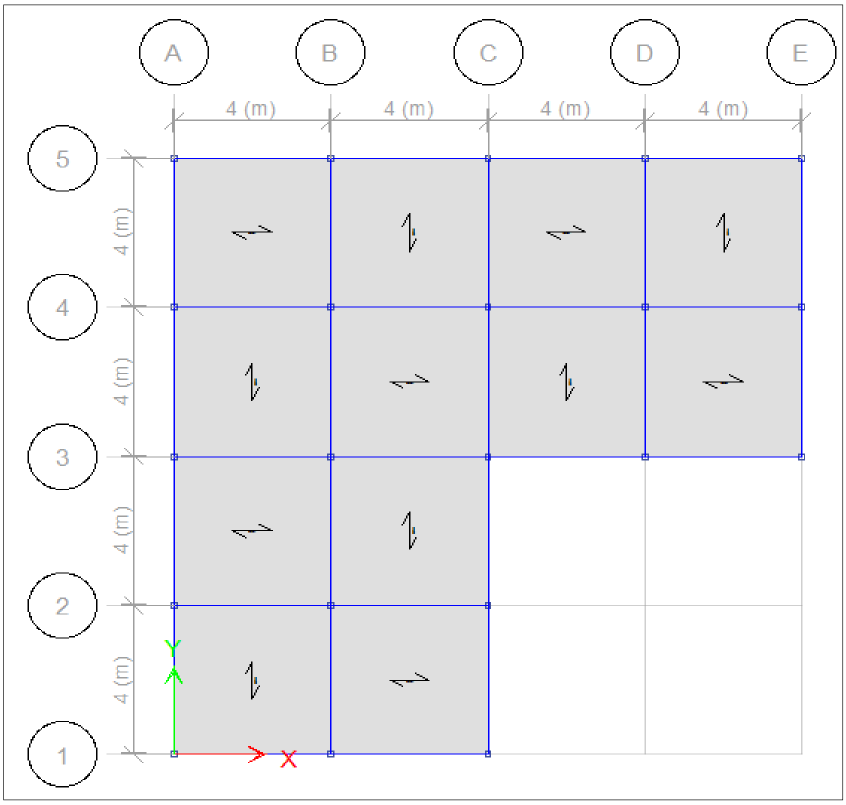


Figure 2. The typical framing plan

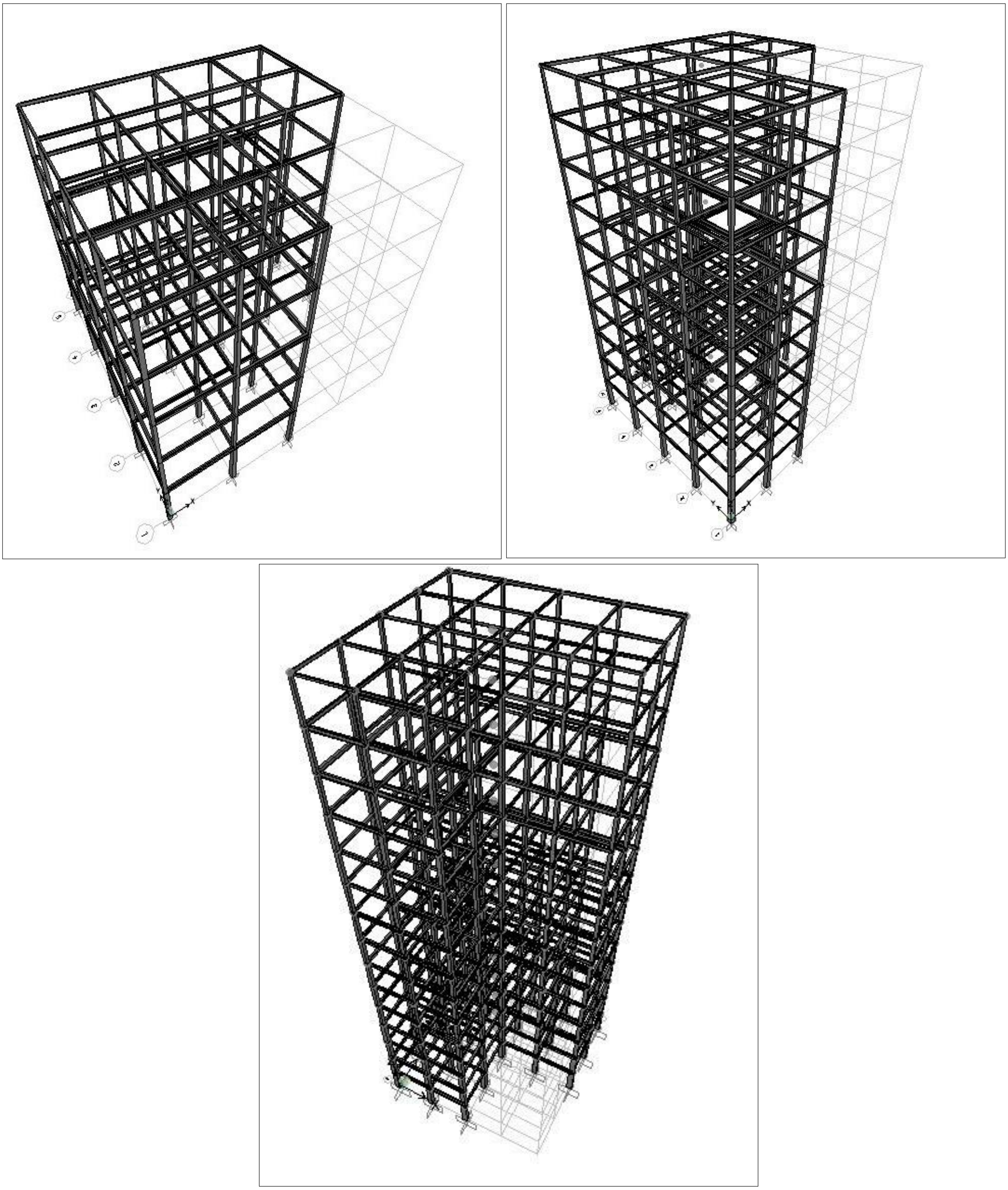


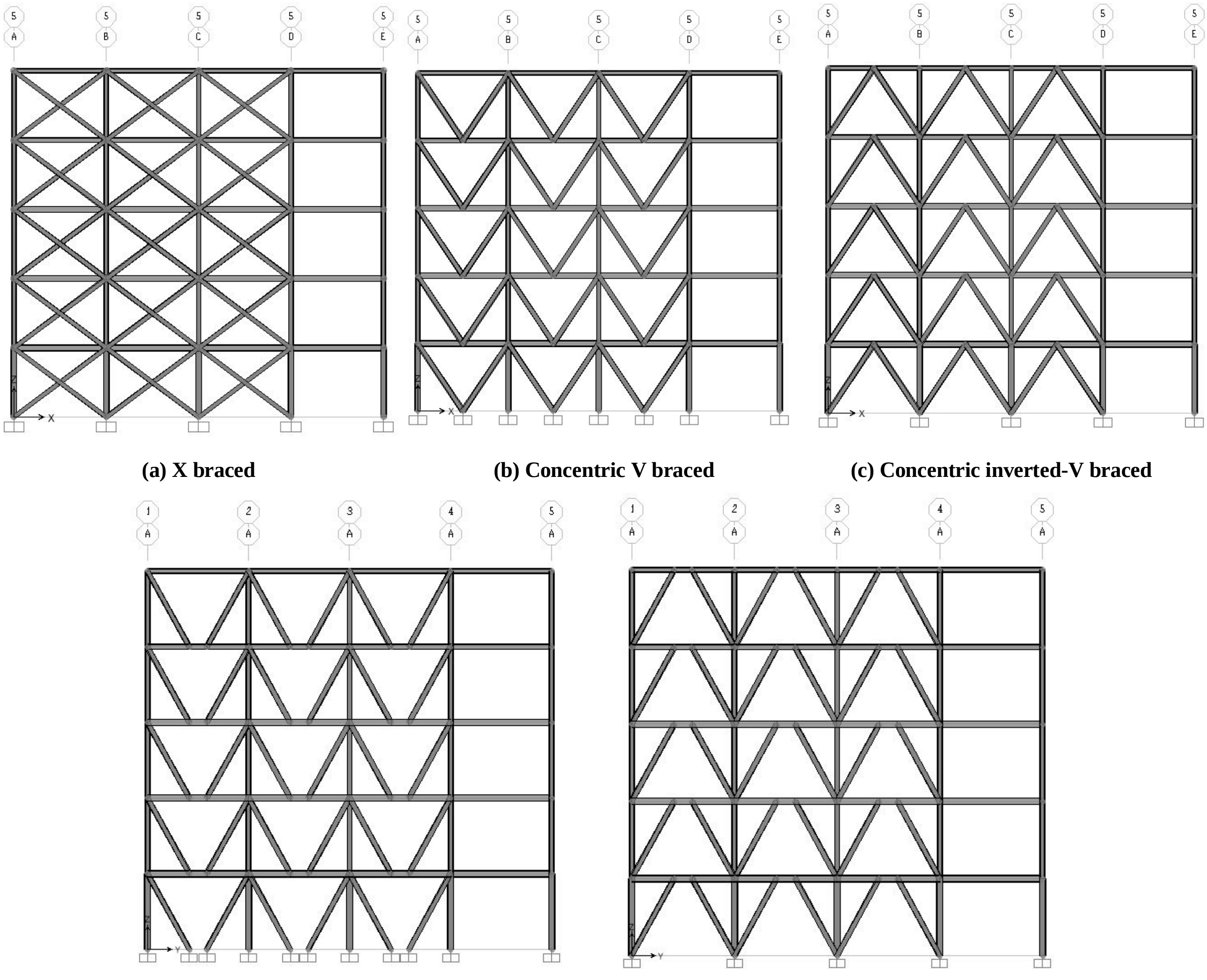
Figure 3. The 3D models of study structures with moment-resisting frames

2.2. Seismic Rehabilitation of Structures

The frames were rehabilitated with five different bracing systems including: X-braced frames, two eccentric braces (chevron V and inverted V), and two concentric braces (Chevron V and inverted V). Table 3 shows the bracing details for the structures. They were designed based on national seismic rehabilitation guidelines (Guideline 360) [33]. In eccentrically braced frames, the link size was set as 50 cm. Figure 4 shows the plan of rehabilitated 5-storey structure, as a sample. After rehabilitation of structures, DI again was evaluated for them and compared with those of moment frames.

Table 3. Specifications of the bracing systems for rehabilitation

Structure	Brace sections
5-storey	2UNP16 (all storeys)
10-storey	2UNP16 (storeys 1-3), 2UNP14 (storeys 4-7), and 2UNP12 (storeys 8-10)
15-storey	2UNP18 (storeys 1-4), 2UNP16 (storeys 5-8), 2UNP14 (storeys 9-12), and 2UNP12 (storeys 13-15)



(a) X braced **(b) Concentric V braced** **(c) Concentric inverted-V braced**

(d) Eccentric V braced **(e) Eccentric inverted-V braced**

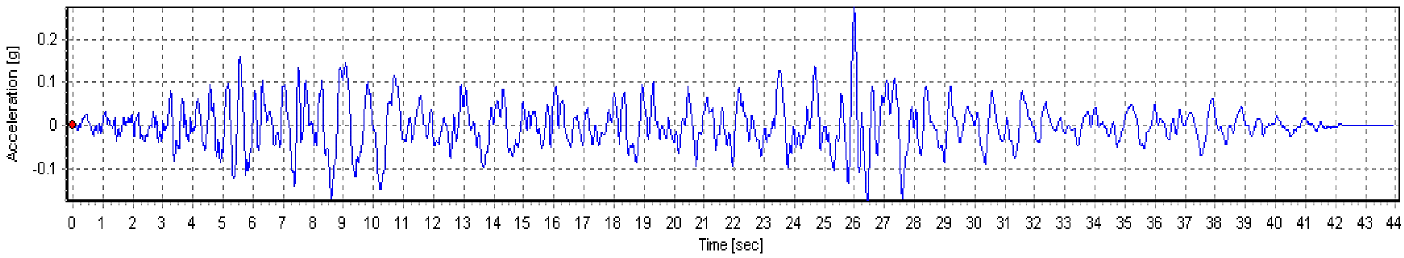
Figure 4. Configurations of rehabilitated models

2.3. Ground Motions

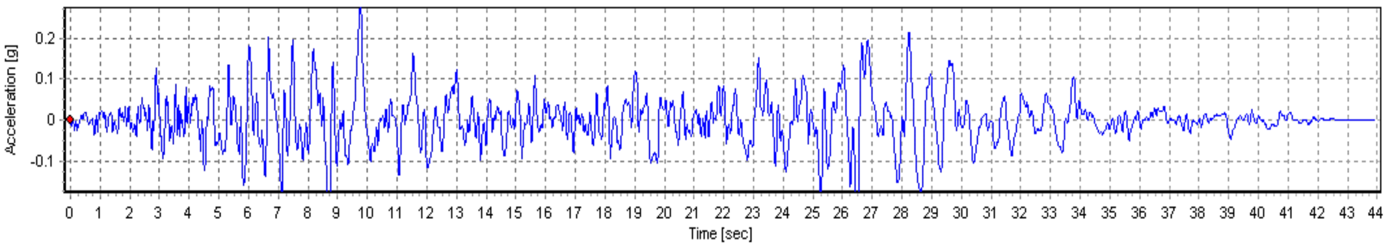
The structures were subjected to five pairs of different ground motions from different countries which were selected from beta version of PEER ground motion database for soil type C condition (see Table 4) which had no pulse signals. The distance of seismic stations from earthquakes was in a range of 10 to 50 km. Figure 5 shows the acceleration time graphs for Landers earthquake in two directions as a sample. Standard 2008 design spectrum was used for scaling these real records. In this regard, scale factor of these records for 5-storey model was obtained as 3.54 and for 10 and 15-storey structures it was calculated as 4.98 based on the design base acceleration of Tehran (A=0.35g) and their SRSS. The average of the SRSS spectrum for all ground motions with 1.3 times the Standard 2008 spectrum was calculated in a period of 0.2T-1.5T (T= Main period of vibration for each frame). It was supposed that the average SRSS spectrum for all records should not fall below 1.3 times the Standard 2008 spectrum in the mentioned period. Figure 6 presents the comparison between the SRSS spectrum of five real records and Standard 2008 spectrum in 5-storey model, as a sample.

Table 4. Earthquake records

Earthquake Name	Year	Station Name	Magnitude	PGA
Landers, US	1992	Joshua Tree	7.28	2.989
Manjil, Iran	1990	Qazvin	7.37	1
Irpinia, Italy	1980	Calitri	6.9	4.9167
San Simeon, US	2003	San Antonio Dam - Toe	6.52	8.5479
Darfield, New Zealand	2010	SPFS	7	4.4705



(a) X direction



(b) Y direction

Figure 5. The accelerograph of Landers earthquake in two X and Y directions

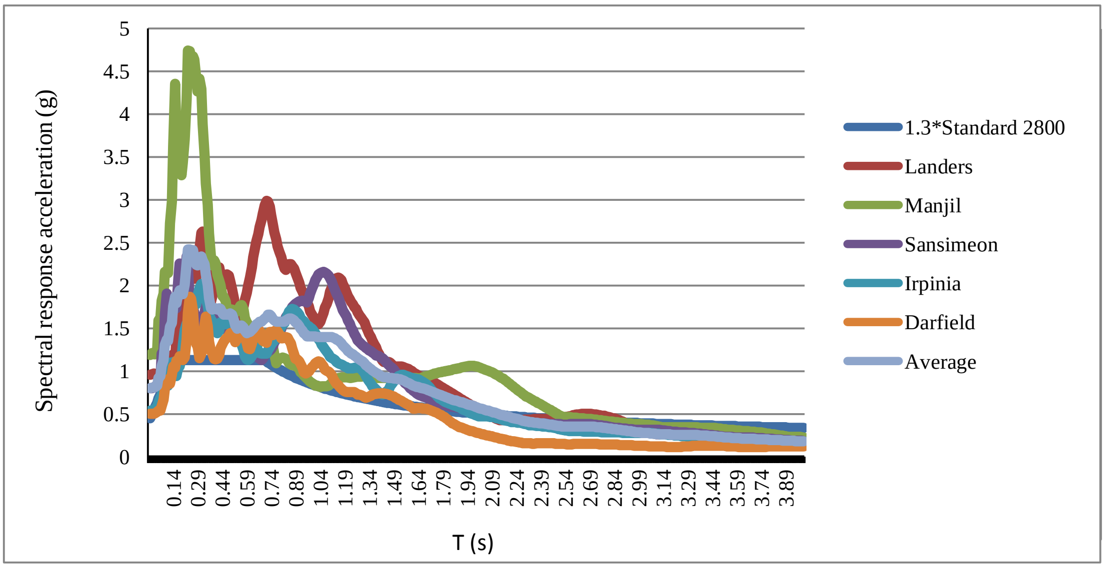


Figure 6. Comparison of the SRSS spectrum of five real records with Standard 2008 spectrum in 5-storey model

2.4. Damage Evaluation

After selecting ground motions, we evaluated damage of moment resisting frames. For this purpose, the response-based damage index of Ghobara et al. [6] was used which is based on stiffness. In this method, damage index (DI) for the *i*-th storey is defined as:

$$DI^{i} = 1 - \frac{K^{i}_{final}}{K^{i}_{initial}} \tag{1}$$

Where;

$K^{i}_{initial}$ = the initial slope of the pushover curve of the i-th storey before subjection to the earthquake and K^{i}_{final} = the initial slope of the pushover curve after subjecting the frame to the earthquakes [6]. The DI for the whole frame is also obtained as following:

$$DI = 1 - \frac{K_{final}}{K_{initial}} \tag{2}$$

In Ghobara damage index, two pushover analyzes are performed for the structure; once before subjecting the structure to the earthquake and once after subjection to the ground motion which is shown Figure 7. The structure is returned to the unloaded static state before performing the second pushover analysis.

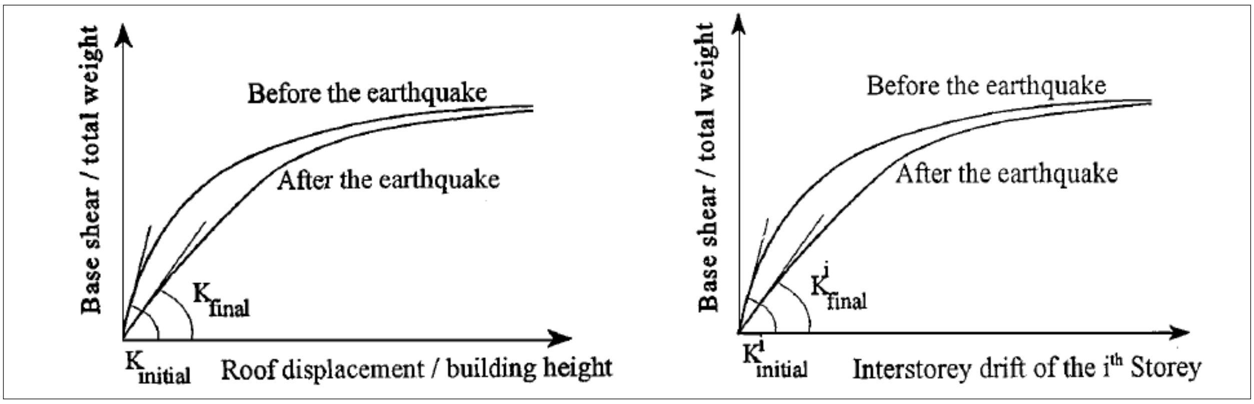


Figure 7.Pushover analyses for measuring DI adapted from [6]

The values of the DIs range from zero to one depending on the amount of experienced damage. A value of zero indicates no damage while value one shows the collapse. The index is not sensitive to the distribution of the applied load in the pushover analysis. The effect of torsion and building irregularities can be shown by the 3D pushover analysis. For more information see Reference [6].

3. Results

3.1. Damage Evaluation of Structures with Moment Resisting Frame

For damage evaluation, first static nonlinear pushover analysis of the frame was performed in SAP 2000 application to determine the initial stiffness of the frame (before subjection to the earthquakes). Second, the frame response to an applied earthquake ground motion was determined. Next the second pushover analysis was performed to determine the stiffness after subjecting to the earthquakes. Then the damage index was calculated for each storey using Equations (1) and (2).

3.1.1. The 5-Storey Structure

In 5-storey structure, due to the symmetry of the structure in the two directions of X and Y, the values of the base shear and displacement obtained for the pushover analysis were the same in two directions. Based on Equations (1) and (2), for obtaining stiffness (K), we can write:

$$K = \frac{\frac{V_2}{w} - \frac{V_1}{w}}{\frac{\Delta_2}{H} - \frac{\Delta_1}{H}} \tag{3}$$

Where;

V_2 and V_1 are first and second base shears of each storey; Δ_2 and Δ_1 represent storey displacements of first and second base shears in pushover analysis; W is the storey weight; and H shows the storey height. Tables 5 and 6 present the results of base shear and displacement for the first storey of 5-storey structure before and after subjecting to the Landers earthquake, as a sample. Using these results, DI index was obtained for the first storey as shown in Table 7. For other storeys and other earthquakes, the DI index was obtained in the same way whose results are shown in Figure 8. Range of damage index proposed in [6] is as following: Minor = 0.0–0.15; Moderate (repairable) = 0.15–0.3; Severe (irrepairable) = 0.3–0.8; and Collapse >0.8. Based on these ranges, maximum DI results for 5-storey building showed that this building was in severe state with a range between 0.3 and 0.5. Only in first storey moderate damage was observed.

Table 5. Base shear and displacement results of the first storey before subjecting the 5-storey structure to the Landers earthquake

Before earthquake					
Shear (Kgf)					
Pushover	V		W	V/W	
	X direction	Y direction		X direction	Y direction
1	0	0	21120	0	0
2	110035	125694		5.20999053	5.951420455
Displacement (m)					
Pushover	Δ		H	Δ/H	
	X direction	Y direction		X direction	Y direction
1	0	0	3	0	0
2	0.0065	0.0074		0.002166667	0.002466667

Table 6. Base shear and displacement results of the first storey after subjecting the 5-storey structure to the Landers earthquake

After earthquake					
Shear (Kgf)					
Pushover	V		W	V/W	
	X direction	Y direction		X direction	Y direction
1	21627	1946	21120	1.0240	0.092140152
2	136848	40053.24		6.4796	1.896460439
Displacement (m)					
Pushover	Δ		H	Δ/H	
	X direction	Y direction		X direction	Y direction
1	0.00673	0.00009	3	0.002243333	0.00003
2	0.02037	0.00456		0.00679	0.00152

Table 7. Measured DI for the first storey of 5-storey structure based on Ghobara's model

	$k_{initial}$(kgf/m)	**k_{final}(kgf/m)**	**DI**	**Max. DI**
X direction	2405	1200	0.5010	0.5010
Y direction	2413	1211	0.4981	

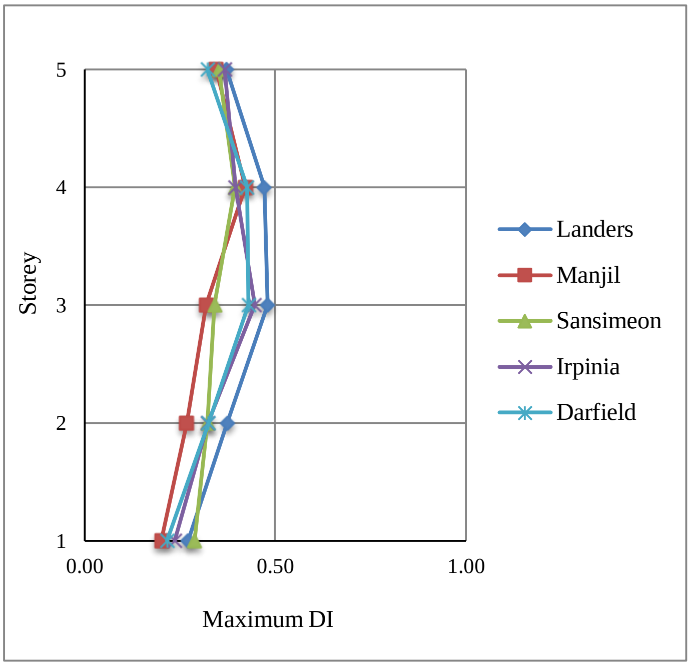


Figure 8. Maximum damage of 5-storey moment frame subjected to the five earthquakes

3.1.2. The 10-Storey Structure

The values of the base shear and displacement obtained for the pushover analysis of 10-storey moment frame in X and Y directions were different from each other before subjection to the earthquakes. The reason for this difference is that we used unequal sections in these two directions. For this structure, base shear, storey displacement and DI were calculated according to the above mentioned procedure. Figure 9 shows the maximum DI results for 10-storey structure which is subjected to the five ground motions. According to the results, under Landers earthquake, storeys 8 and 9 were collapsed (maximum DI > 0.8). A collapse also was observed in storey 8 under Irpinia earthquake. In other storeys and earthquakes severe damage was mostly occurred.

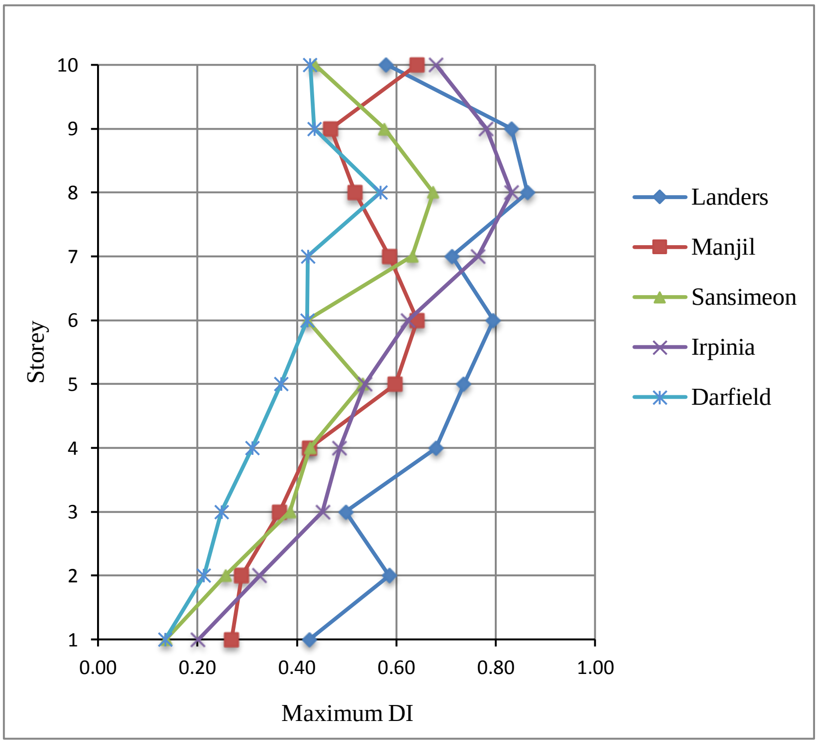


Figure 9. Maximum damage for 10-storey moment frame subjected to the five earthquakes

3.1.3. The 15-Storey Structure

In 15-storey structure before subjection to the earthquakes, the values of the base shear and displacement obtained for the pushover analysis of were also different in X and Y directions due to the use of unequal sections. Maximum DI results for 15-storey structure with moment resisting frame after subjection are presented in Figure 10. Based on the results, it was observed that under Manjil, Irpinia, and Darfield earthquakes, severe damage was occurred in all storeys. Under Landers earthquake, storeys 5, 6, 8, and 9 were collapsed and other storeys were in severe damage conditions. Under San Simeon earthquake, storeys 4, 5, 8, and 14 had moderate damage state and in other storeys damage was in severe condition. Under other three earthquakes severe damage state was observed in all storeys of the 15-storey moment frame.

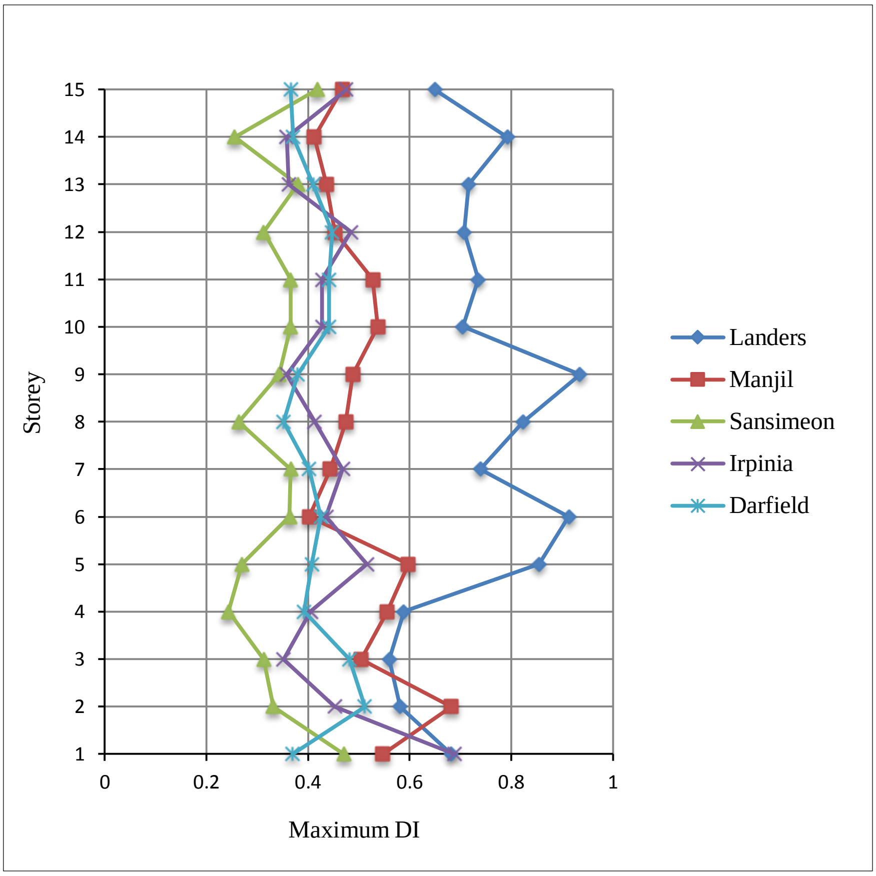


Figure 10. Maximum damage of 15-storey moment frame subjected to the five earthquakes

3.2. Damage Evaluation of the Rehabilitated Structures

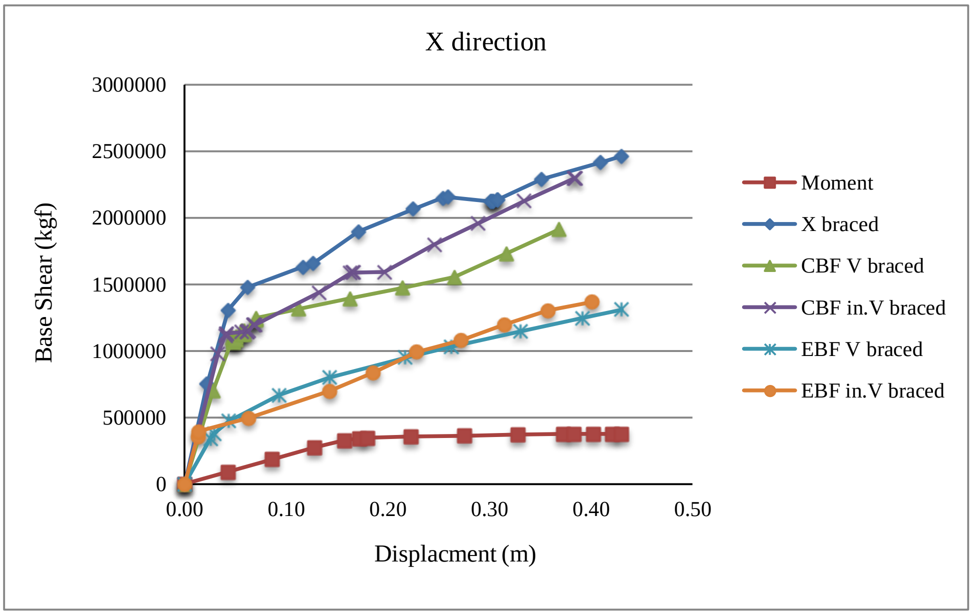
In this section we present the pushover curves for the three rehabilitated structures and their comparison with those of moment resisting frames designed based on the first edition of Standard 2800 [32]. Then, maximum DI results for the rehabilitated structures are illustrated.

3.2.1. The 5-Storey Structure

Figure 11 presents the pushover curves of 5-storey structure rehabilitated with five types of bracing systems in X and Y directions before subjection to the earthquakes, and their comparison with moment frames. As can be seen, the use of braced frames has a significant effect on increasing the amount of base shear absorbed by the structure. With the increase of force and the entrance of members into a plastic hinge region, we see an improved behavior in the pushover curve of the 5-storey structure rehabilitated with five bracing systems in both directions. Also, it was observed that in the nonlinear region, the slope of the pushover curve of the moment frame was less than that of the other rehabilitated frames which indicates the lower stiffness of the moment resisting frame in the nonlinear region. Based on the results, the initial stiffness of X and CBF bracing systems are higher than of EBF systems. On the other hand, their secant stiffness was mostly equal. The reason is related to the plastic hinges formation such that by increasing the lateral displacement of structure, the plastic hinges were formed in the beams and columns of moment frames while there was almost no yielding occurrence in the braced frames. The values in Table 8 are extracted from pushover curves after idealization. According to the results, yield stress in X-bracing system was higher than that of other bracing systems where moment resisting frame had the lowest yield stress (349105 Kgf). Also, the highest yield displacement was related to the structure with moment resisting frame (16 cm). By comparing behaviour factors of braced frames, it was found that 5-storey structure rehabilitated with concentric V-braced system had higher value of behaviour factor compared to other frames (9.88) where X-braced frame showed the least ductility with lower behavior factor.

Table 8. Behavior factor of study frames in 5-storey structure

Frames	Yield stress (Kgf)	Yield displacement (cm)	Behaviour factor
Moment	349105	16	4.67
X braced	1527975	7	8.81
CBF V braced	1249270	5	9.88
CBF in. V braced	1225732	5	9.12
EBF V braced	487510	4	9.10
EBF in. V braced	487543	4	9.57



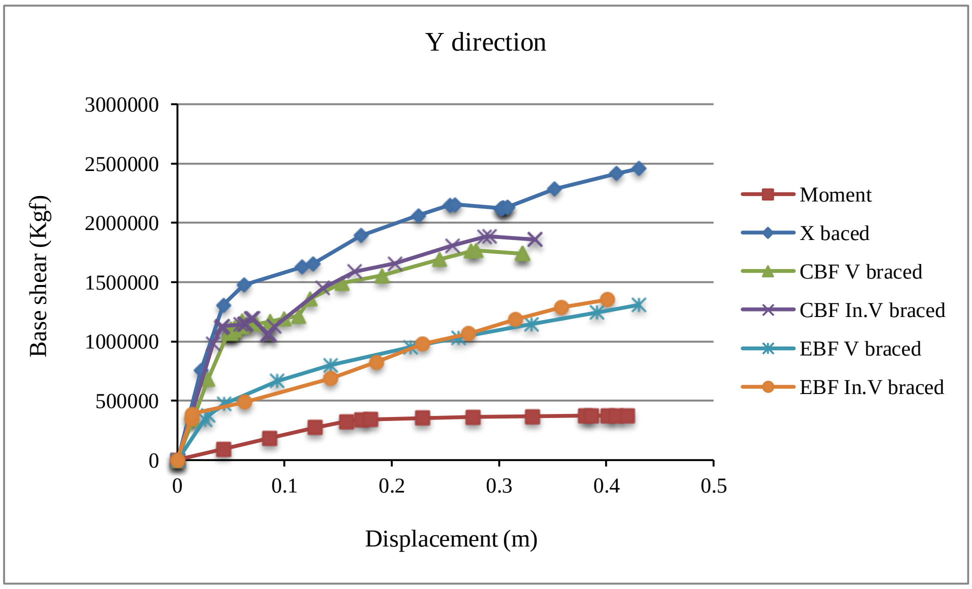


Figure 11. Pushover curve for 5-storey structure rehabilitated with five bracing systems in X and Y directions

Maximum damage of rehabilitated 5-storey structure was assessed for each bracing configurations using Equations (1) to (3). The results are shown in Figure 12. Based on Guideline no.360 for buildings with irregular plan, results indicated a significant reduction of damage rate. The maximum damage index of this structure was less than the acceptable rate of minor damage reported in [6] (DI = 0.15). It was observed that under all bracing configurations, damage resulted from all earthquakes turned from severe state into minor state in all storeys. With increasing number of storeys, the damage rate for the 5-storey structure rehabilitated with X-braced and concentric inverted-V braced frames was increased, while for eccentric V and inverted-V types, it was decreased. For concentric V bracing system, by increasing number of storeys, damage amount can be either increased or decreased.

By comparing the behavior of braced frames with five different configurations in 5-storey structure based on maximum damage obtained using Ghobara DI, it was found out that, in average, X-braced frames with the least damage had the best seismic performance with a 95% improvement. CBF systems behaved better than EBF systems in 5-storey structure; concentric V and inverted-V bracing systems showed 94 and 92% improvement respectively, while for eccentric V and inverted-V types it was 90 and 85%, respectively.

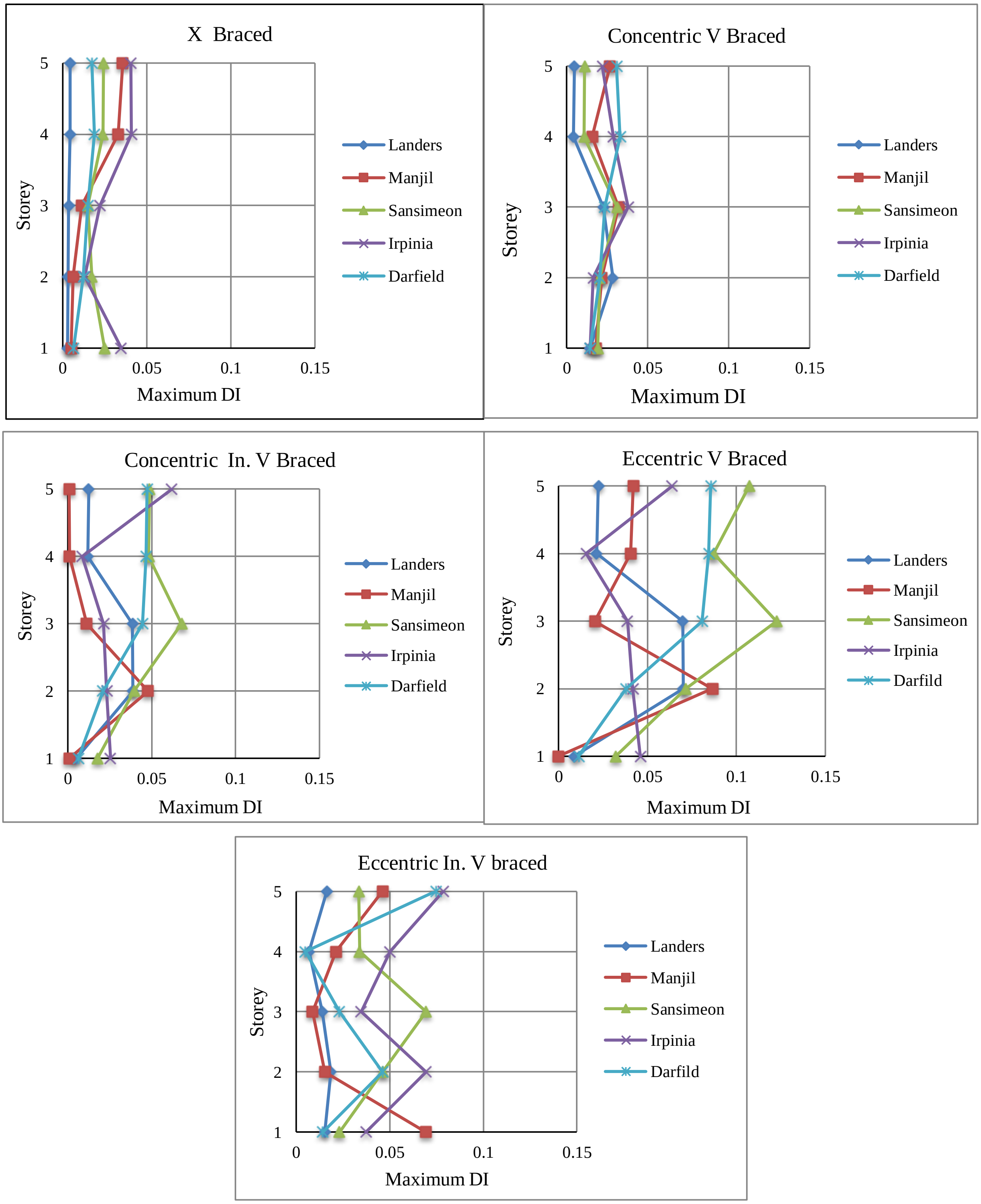


Figure 12. Maximum damage of 5-storey structure rehabilitated with five bracing systems under five earthquakes

3.2.2. The 10-Storey Structure

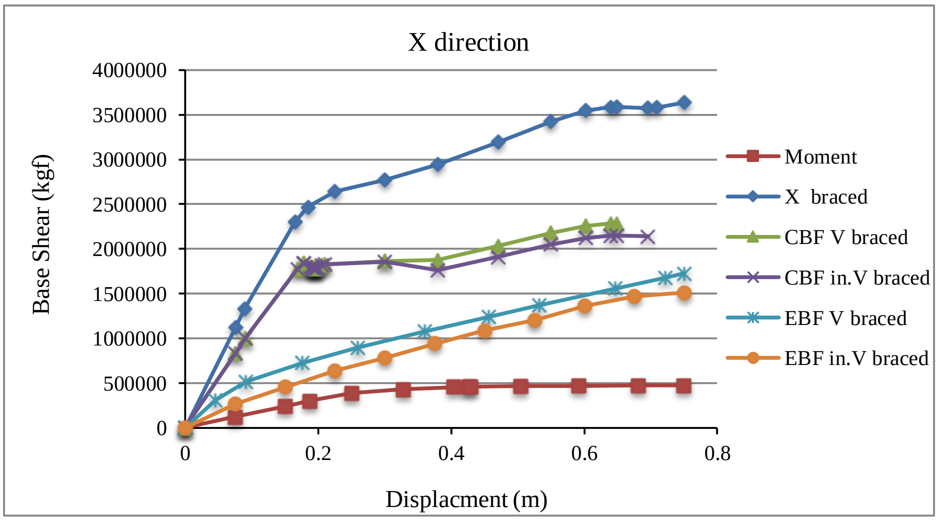
For the rehabilitated 10-storey structure, pushover curves in X and Y directions before subjection to the earthquakes are shown in Figure 13. As can be seen, braced frames have increased the amount of base shear absorbed by the structure. An improved behavior in the pushover curve of the 10-storey structure rehabilitated with five bracing systems was observed in both directions with the increase of force and the entrance of members into a plastic hinge region. The slope of the pushover curve of the moment frame was less than that of the other rehabilitated frames. Similar to 5- storey structure, initial stiffness of X and CBF bracing systems were higher than that of EBF systems, while their secant stiffness was almost equal. Based on the results in Table 9, yield stress in X-bracing system was higher than that of other bracing systems where moment resisting frame showed the lowest yield stress (435295 Kgf). Also, the yield displacement of the 10-storey structure with moment resisting frame was higher than that of structure with braced frames (27 cm). Moreover, 10-storey structure rehabilitated with eccentric inverted V-braced system had higher value of behaviour factor compared to other rehabilitated 10-storey structures (7.88) where X-braced frame was reported as the least ductile system.

Table 9. Behavior factor of study frames in 10-storey structure

Frames	Yield stress (kgf)	Yield displacement (cm)	Behavior factor
Moment	435295	27	4.07
X braced	2546698	18	7.17
CBF V braced	1871370	17	7.41
CBF in. V braced	1803111	17	7.74
EBF V braced	801234	17	7.61
EBF in. V braced	803144	16	7.88

Maximum damage index of rehabilitated 10-storey structure (Figure 14) showed significant effect of five bracing systems on seismic performance of the structure. Under all bracing systems, damage resulted from all earthquakes turned from severe state into minor state in all storeys. Similar to 5-storey structure, with X-braced and concentric inverted-V braced frames, the damage rate for the 10-storey structure was increased with increasing number of storeys, while for eccentric V and inverted-V types, it was decreased. The increase in number of storeys in concentric V braced frames either increased or decreased the damage rate.

Based on the results, in rehabilitated 10-storey structure, eccentric inverted-V bracing system had the highest effect on seismic behavior compared to moment resisting frame which improved the seismic performance of the structure by 95%. After this type, X-braced, eccentric V-braced, concentric inverted-V braced, and concentric V braced frames showed the highest rate of performance improvement as 94, 92, 91 and 88% , respectively compared to moment resisting frame.



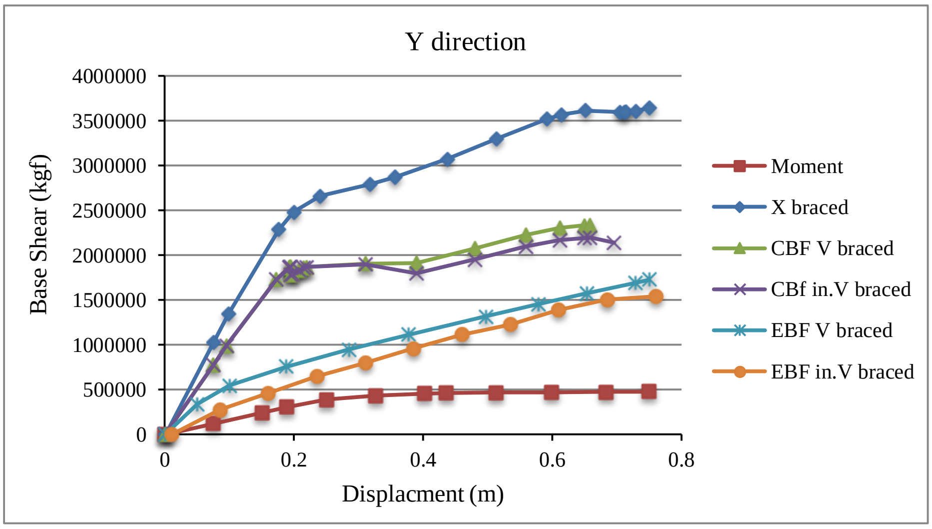
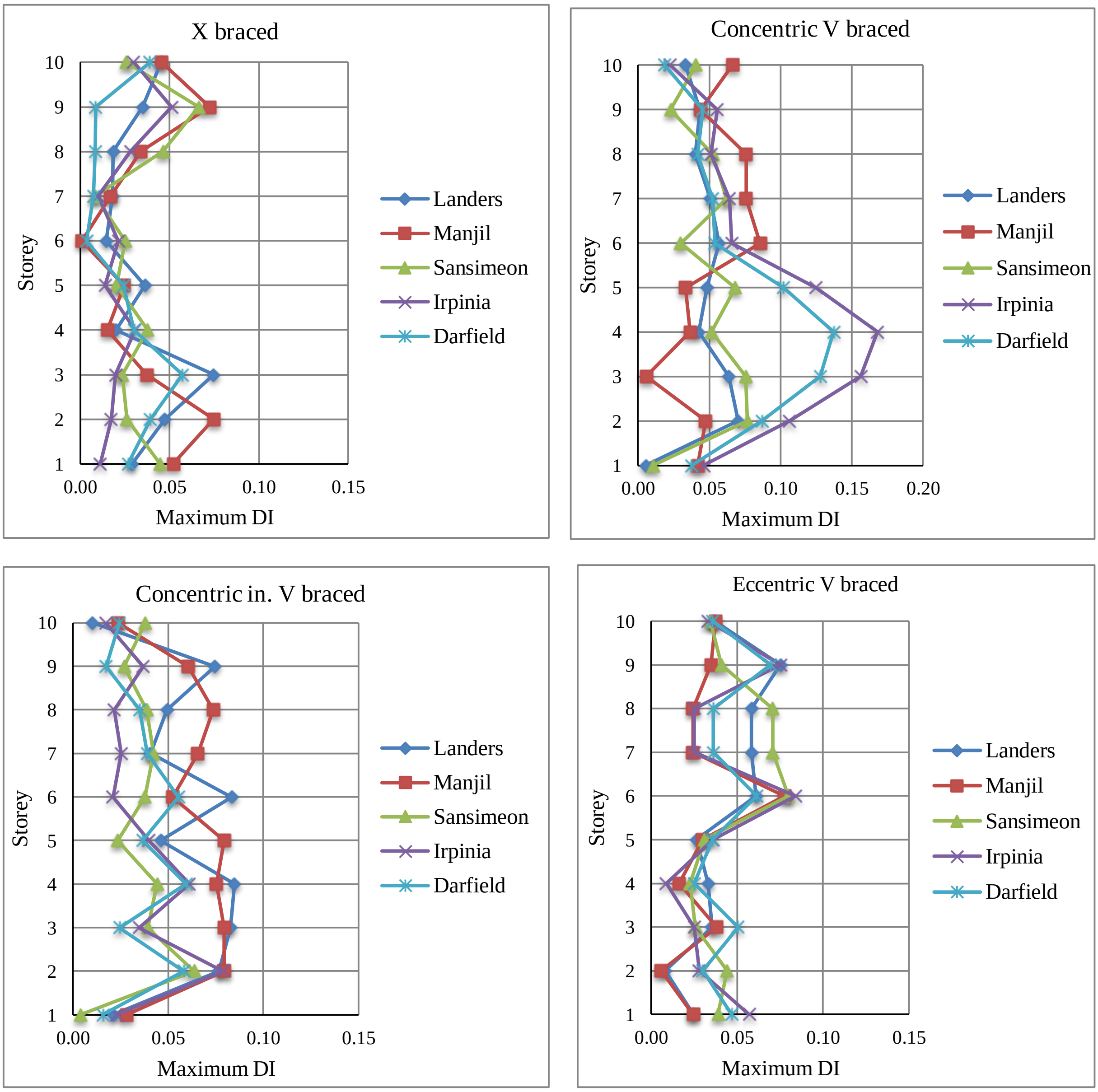


Figure 13. Pushover curve for 10-storey structure rehabilitated with five bracing systems in X and Y directions



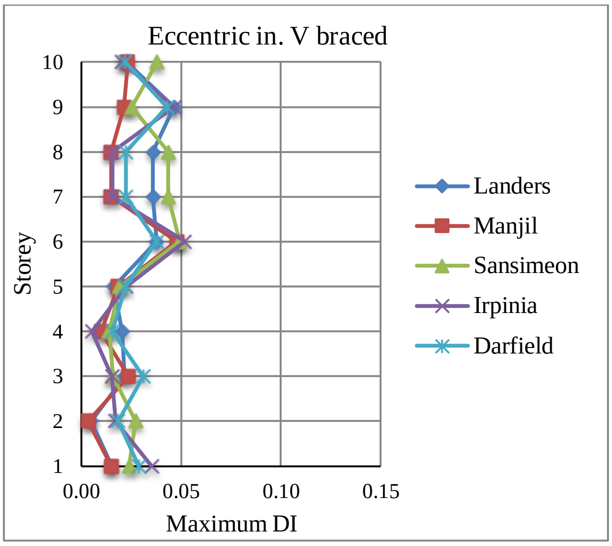


Figure 14. Maximum DI of 10-storey structure rehabilitated with five bracing systems under five earthquakes

3.2.3. The 15-Storey Structure

Figure 15 presents pushover curves in X and Y directions for the rehabilitated 15-storey structure before subjection to the earthquakes. It can be seen that the the base shear absorbed by the structure has been increased by using braced frames and they can suffer more base shears. Hence, we can say that braced frames have improved the seismic behavior of 15-storey structure. Also, results indicated the lower stiffness of the moment resisting frame in the nonlinear region, since the slope of the pushover curve of the moment frame was less than that of braced frames. Similar to 5 and 10-storey structures, initial stiffness of X and CBF bracing systems were higher than that of EBF systems, while their secant stiffness was almost equal. Based on the results in Table 10, yield stress in X-bracing system was higher than that of other bracing systems where moment resisting frame had the lowest yield stress (539061 Kgf). Also, the 15-storey structure with moment resisting frame had the highest yield displacement compared to the structures with braced frames (38 cm). Moreover, the 15-storey structure rehabilitated with EBF systems had higher value of behaviour factor compared to other rehabilitated 15-storey structure; where behaviour factor of the structure with X-braced frame was lower (3.86). The reason for this is that with increasing storey height and roof displacement, this braced frame suddenly collapses. This results in a sudden decrease in the pushover curve; hence the behavior factor decreases. But in other bracing systems, the link increases the ductility.

Table 10. Behavior factor of study frames in 15-storey structure

Frames	Yield stress (Kgf)	Yield displacement (cm)	Behaviour factor
Moment	539061	38	3.86
X braced	2652134	35	5.47
CBF V braced	2085246	28	5.62
CBF in. V braced	1927845	28	6.60
EBF V braced	967168	23	6.17
EBF in. V braced	961047	23	6.07

Results of maximum DI for rehabilitated 15-storey structure (Figure 16) showed that, by using braced frames, the severe damage state of the all storeys reduced to minor state which indicates the better behavior of steel bracing systems under earthquakes compared to moment resisting frames. Generally, the results of increasing number of storeys on the DI of the rehabilitated 15-storey structure are the same as the rehabilitated 5 and 10-storey structures.

In average, eccentric V and inverted-V bracing systems showed the highest effect on the seismic behavior of the rehabilitated 15-storey structure where improvement rate was 96%. The next top ranking systems were concentric V, concentric inverted-V and X-bracing which improved the seismic behavior of 15-storey structure by 92, 88, and 85%, respectively.

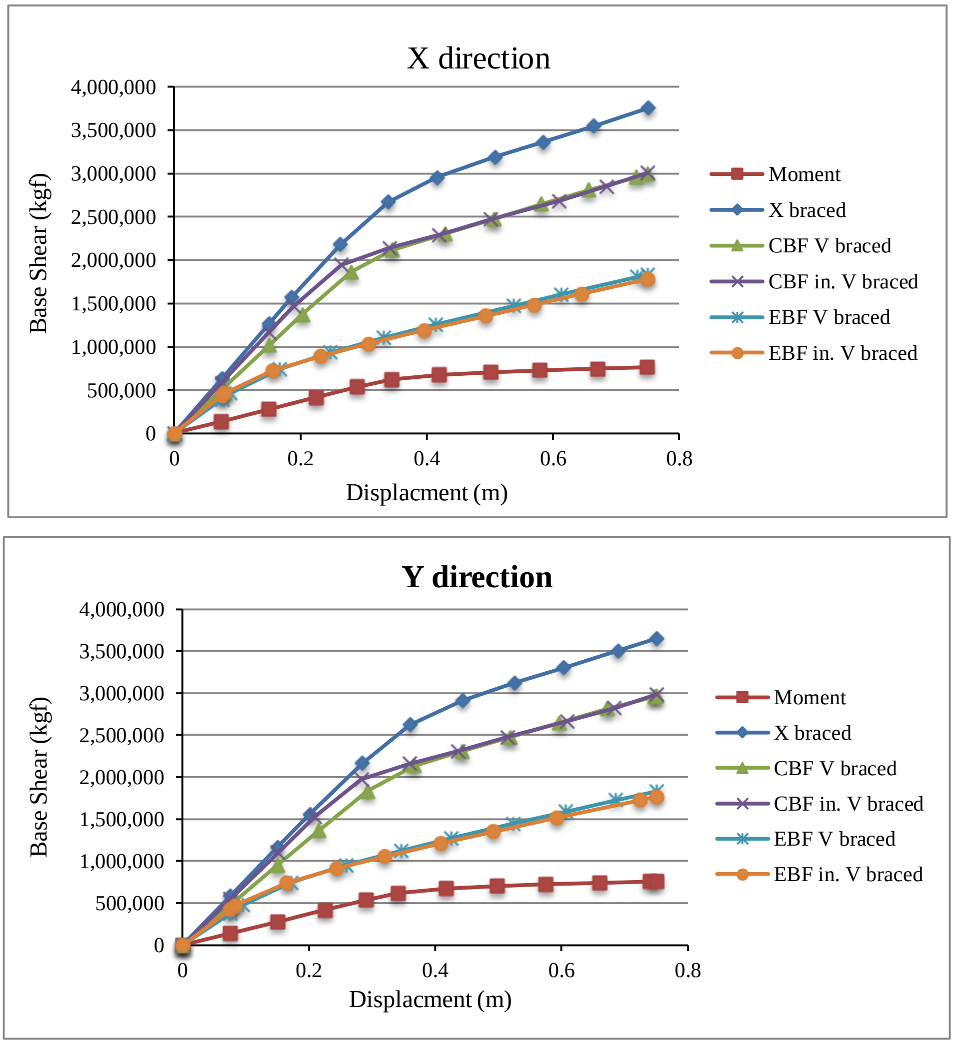
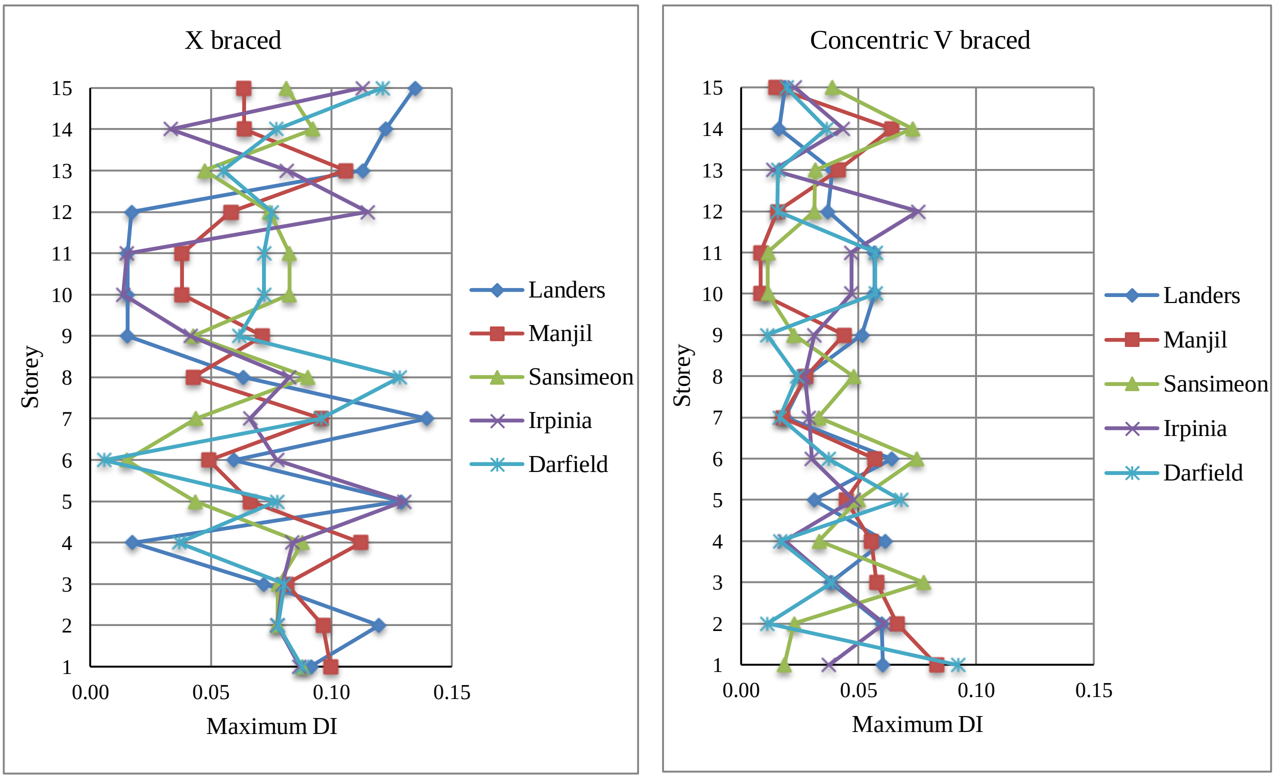


Figure 15. Pushover curve for 15-storey structure rehabilitated with five bracing systems in X and Y directions



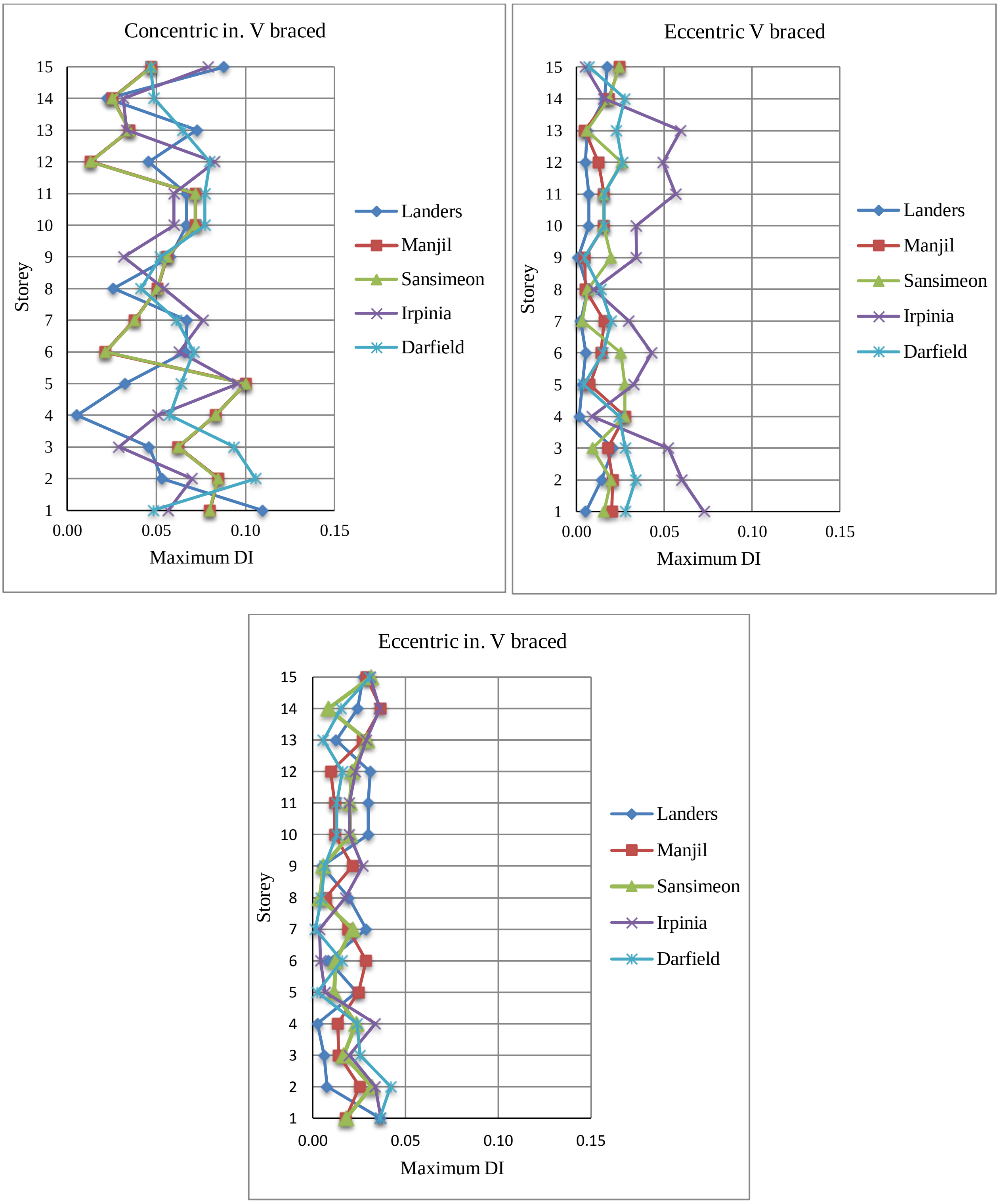


Figure 16. Maximum DI of 15-storey structure rehabilitated with five bracing systems under five earthquakes

4. Conclusion

In this paper, three 5-, 10- and 15-storey steel buildings with moment resisting frames were rehabilitated with five different bracing systems. To compare the bracing systems and to know which of these systems had a better effect on the structure performance, response-based damage model in [6] which is based on stiffness, was used. The structures subjected to five earthquakes and then their results were analyzed by conducting pushover analysis. Based on the results of pushover analysis, it was found out that among studied bracing systems, the X-bracing system had the lowest ductility with lower value of behavior factor, but its initial stiffness and yield stress were higher than that of other study systems.

In rehabilitated low-rise building, X-braced frames with the least damage showed best seismic performance with a 95% improvement. CBF systems behaved better than EBF systems where concentric chevron V and inverted-V bracing systems showed 94 and 92% improvement respectively, while for eccentric V and inverted-V types it was 90 and 85%, respectively.

In rehabilitated mid-rise building, EBF system with chevron inverted-V configuration had the highest effect on seismic behavior of the structure compared to moment resisting frame which improved the seismic performance of the structure by 95%. After this type, X-braced system, EBF with chevron V-type configuration, and CBF with Chevron inverted-V and V models showed the best performance with 94, 92, 91, and 88% improvement, respectively compared to moment resisting frames.

In rehabilitated high-rise building, EBF systems showed the highest effect on the seismic behavior of the rehabilitated 15-storey structure where improvement rate was 96%. The CBF systems (chevron V and inverted-V) and X-braces improved the seismic behavior of 15-storey structure by 92, 88, and 85%, respectively.

Results showed that with increasing number of storeys, the damage amount for the structures rehabilitated with X-braced and concentric inverted-V-braced frames was increased such that the maximum damage of mid- and high-rise buildings with these bracing systems were reported as 1.73 and 4.17, and 1.63 and 2.09 times the damage observed in low-rise building with the two frames, respectively. Hence, it can be said that the seismic performance of these two braced frames decreases with the increase of storey height. For eccentric V and inverted-V types, the damage amount decreased when the number of storeys increased. In mid- and high-rise buildings with eccentric V and inverted-V bracing systems, the failure rate was reduced as 77 and 37%, and 72 and 56%, respectively compared to low-rise building. Therefore, we can say that EBF systems' performance increases by increasing storey height. The efficiency of concentric V bracing system was not constant with increasing height and it can be either increased or decreased.

Based on the results, we concluded that, although the use of nonlinear dynamic and static (pushover) analyses is proper solution for seismic retrofitting of structures, but with these methods it is not possible to detect the amount of damage, therefore, the use of response-based damage indices can be a suitable procedure for estimating the failure rate of steel structures.

We recommend further studies on the evaluation of steel structures with regular plans and the effect of retrofitting methods on these structures and their differences with irregular plans. Also, other damage models are suggested to be used in further studies to evaluate the vulnerability of steel frames as well concrete frames.

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