Civil Engineering Infrastructures Journal, 50(2): 353 – 374, December 2017
Print ISSN: 2322-2093; Online ISSN: 2423-6691
DOI: 10.7508/ceij.2017.02.009

Evaluation of Performance Levels of Zipper-Braced Frames Using Structural Damage Index

Vaseghi Amiri, J.[1], Esmaeilnia Amiri, M.[2*] and Ganjavi, B.[3]

[1] Professor, Faculty of Civil Engineering, Babol University of Technology, Babol, Iran.
[2] Assistant Professor, Department of Civil Engineering, Ayatollah Amoli Branch, Islamic Azad University, Amol, Iran.
[3] Assistant Professor, Department of Civil Engineering, University of Mazandaran, Babolsar, Iran.

Received: 17 Mar. 2017; Revised: 03 Aug. 2017; Accepted: 11 Aug. 2017

ABSTRACT: The determination of structural and nonstructural damage under earthquake excitations is usually considered as a key factor in performance-based seismic design (PBSD) methods is In this regard, various damage indices have been developed in recent years to quantitatively estimate structural damage. The aim of this study is to develop a simple method to evaluate performance levels of zipper-braced frame (ZBF) structures by using damage indices based on the results of nonlinear static and dynamic analyses. To this end, 5, 7, 10, 12 and 15 story zipper-braced frames (ZBF) are modeled and undergone to twenty different synthetic ground motion records and their damage values have been computed. In dynamic damage analysis procedure, the performance levels of the ZBF models have been computed based on the FEMA-356 standard. Considering the results of the nonlinear dynamic analyses, the correlation between FEMA-356 performance levels and damage indices has been investigated and some simplified formula is presented. On the other side, in static damage analysis approach, by using pushover analysis the performance points of ZBF models have been estimated based on capacity spectrum method (CSM) provided by ATC-40 standard. Then, the correlation between ATC-40 performance levels and some static damage indices has been investigated and some simple equations have been proposed. These relations can be utilized to estimate the performance levels of structures from damage indices. Finally, tables are represented for determination of the structural damage index values for assumed performance levels of the ZBF structures based on static and dynamic damage analysis.

Keywords: ATC-40, Damage Indices, FEMA-356, Performance Levels, ZBF Structures.

INTRODUCTION

During the recent earthquake events, the structures which were designed in accordance to the new seismic design philosophy have shown an appropriate behavior in the performance level of life safety; however, the amount of structural and nonstructural damage as well as its economic losses in the structures was unexpected. As a result, predicting the amount of structural damage at different hazard levels can be considered as one of the most important subjects in PBSD. In performance-based design methods,

* Corresponding author E-mail: m.e.amiri@Iauamol.ac.ir

structures should satisfy the performance requirements under different levels of seismic demand. Therefore, it is necessary to quantify damage under different levels of seismic ground motion. A damage model is an analytical formula to calculate degradation in structures. In general, to quantify the amount of damage in building structure, an index is calculated damage that called a damage index, *DI*. Many different damage indices have been introduced during the past two decades to determine the structural damage. Each of them employs different parameters to quantitatively measure the amount of structural damage. These parameters include plastic deformations, dissipated energy of elements, cyclic fatigue and variation of dynamic characteristics of structure such as fundamental period. Damage indices vary from 0, indicating no damage, to 1 indicating total collapse or failure. Damage indices may be local, for structural elements, or global, for a whole structure. On the other hands, a local damage index is an index to indicate the damage imparted to an element or a story, whereas a global index exhibit an estimate of total damage to the structure. Williams and Sexsmith (1995) showed that the most local damage index parameters are cumulative factors in reality that reflect the dependence of damage on both the amplitude and the number of loading cycles. The main disadvantages of almost all local damage index parameters are essential for adjusting the coefficients for a considered structural system and the lack of calibration against various severities of damage. Global damage indices may be calculated by defining a weighted average of the local index in the entire elements of structure, or by comparing the modal properties of the structure before and after undergoing the earthquake. Although, the global damage indices have less accuracy than local damage index, it is useful to evaluate the behavior of the whole structure without performing heavy computations and analyses. Therefore, in this paper, calculation of the total damage index by means of the global damage indices has been investigated.

During the past decades, extensive studies have been conducted on vulnerability assessment by providing the various damage models. Bertero and Bresler (1971) presented the local, global and cumulative vulnerability definitions for buildings and proposed the vulnerability assessment method by using the static analysis of structures. Banon and Veneziano (1982) proposed a damage model based on the maximum displacement, failure displacements and hysteretic energy dissipation. Park and Ang (1985) proposed a damage index as a linear combination of the damage resulted from significant relative displacement and the effect of repeated cyclic loading. Powell and Allahabadi (1988) defined structural damage in terms of plastic ductility. Bracci et a1. (1989) define a damage index for structural members using a linear combination between ductility ratio and dissipated energy parameter. Dipasquale and Cakmak (1990) proposed a structural damage index for RC structures as the degree of stiffness degradation, involving the ratio between natural period for undamaged and that for the damaged structure during cycle loading. Fardis (1994) suggested an energy based damage index which is a modification of Park-Ang index that the maximum rotation is substituted by the peak value of the member deformation energy. The structural response and low-cycle fatigue effects is considered by Reinhorn and Valles (1995) damage index. Usami and Kumar (1998) developed a model to estimate damage index in steel bridges capable by taking the large deformation into account. Ghobarah et al. (1999) introduced a damage stiffness index for evaluation of seismic performance of RC MRFs. This approach was based on the static pushover analysis and to estimate the expected damage to structures when undergone to earthquake excitations. Bozorgnia and Bertero (2001a, 2001b) introduced two improved damage indices and their corresponding damage spectra to quantify damage potential of the strong ground motions. The improved damage spectra clearly satisfied the structural performance definitions at the limit states of being 0 and 1. Colombo and Negro (2005) proposed an index corresponded to strength loss. They adopted a new method which is based on the actual value of yield moment or force and the value characterizing the yield point in the theoretical skeleton curve. Jeong and Elnashai (2007) presented a new three dimensional damage index for RC buildings that considered the bidirectional and torsional response effect in the 3D RC structures with planar irregularities. Poljansek and Fajfar (2008) have been conducted another research about reinforced concrete frame structures. They take into account the deformation capacity deterioration due to the low cycle fatigue effect and proposed a new damage model for RC buildings. This damage index is the capability to combines deformation and energy quantities at the element level in order to consider the cumulative damage. Rodriguez and Padilla (2009) proposed a damage index for the dynamic analysis of RC members utilizing the plastic energy dissipated by a structural member and a drift ratio corresponded to failure in the structure. Nazri and Alexander (2012) defined a global damage index as the number of plastic hinges divided by the total plastic hinges required to provide a complete strong-columns weak-beams philosophy. Kamaris et al. (2013) proposed a new damage index for plane steel frames under earthquake ground motion. This index was defined at a section of a steel member and takes into account the interaction between the axial force and bending moment. Rodriguez (2015) used a damage index for a family of 11 ground motions records. Some basic parameters of the response of a SDOF system including the maximum hysteretic energy per unit mass were considered in this damage index. Rajeev and Wijesundara (2014) suggested a new energy based damage index that takes into account the number of inelastic cycles (i.e., total energy dissipated by the structural systems) for concentrically braced frame (CBF). The proposed damage index was compared with commonly used drift based index. The results show that the correlations between the energy based and drift based indices was very high in minor damage levels; however, the correlation decreases with increasing the level of damage due the effect of the number of inelastic cycles. Abdollahzadeh et al. (2015) conducted a research on seismic fragility assessment of special truss moment frames (STMF) using the capacity spectrum method. The results show that significant damage is achieved for mid- and tall-special truss moment structures with a Vierendeel middle panel, due to the buckling and early fracture of truss web members. Also, special truss moment structures with an X-diagonal middle segment indicate a low seismic capacity that leads to considerable expected damage. In another study, Shahraki and Shabakhti (2015) presented an algorithm to model uncertainties in structural component level in order to estimate the performance reliability of RC structures. They concluded that their proposed algorithm can appropriately estimate the adequacy of the performance of RC structures at various damage levels for the structural elements.

Several methods to determine damage value at global levels have been proposed so far. In general, these methods can be divided into four categories involving the following structural demand parameters: stiffness degradation, ductility demands, energy dissipation, and strength demands. One of the most well-known and frequently used indices to assess the structural damage and performance levels of structures is story drift ratio. This index obtained from the maximum relative displacement between two stories normalized to the story height. Ghobarah et al. (1999) showed that some damage indices such as inter-story drift does not account for Influences of cumulative damage due to repeated inelastic deformation and it is considered for traditional damage assessment. Hancock and Bommer (2006) found that that the structural performance of buildings subjected to long duration ground motions is not adequately characterized through maximum inter-story drift. According to Rajeev and Wijesundara (2014) study, the correlation between the energy based and drift based indices decreases with increasing the level of damage due to the effect of the number of inelastic cycles. It seems that inter-story drift cannot be an accurate index to evaluate the performance level of structures subject to ground motions. Some researchers tried to provide a relationship between damage indices and drift index to consider the effect of inelastic cycles under earthquakes. Arjomandi et al. (2009) found that there is a correlation between each structural performance level and its corresponding damage to the structure. They investigated the performance levels of the steel moment-resisting frame structures estimated on the basis of the FEMA-356 (2000) and the values of damage indexes. Elenas (2013) evaluated the correlation between the parameters of seismic intensity and the damage index of structures. He used the Park-Ang model and the drift ratio as damage index and found that the spectral and energy parameters exhibit strong correlation to the damage indices. Habibi et al. (2013) developed a practical damage criterion based on pushover analysis. They proposed a simple and effective index to quantify the amount of damage to the structure on the basis of the numerical results of nonlinear static analysis. Recently, Nazri and Alexander (2014) conducted a study on SDOF and MDOF models and found that there is a general trend correlation between drift and their predefined damage index.

In this paper, in order to determine the performance level of ZBF structures, two damage analysis approaches involving nonlinear static and dynamic analyses have been considered. To evaluate damage indices values under earthquake excitations, several important global damage indices have been selected and the correlation between these damage indices and FEMA-356 (2000) damage criteria has been investigated and the performance level of ZBF systems is obtained. Finally, a table is developed to quantitatively determine the performance levels of ZBF frames based on FEMA-356 (2000) criteria from the results of damage indices. The analytical results of this study indicate that there is an appropriate correlation between theses damage indices and FEMA-356 (2000) damage criteria. In addition, to obtain the damage values by means of static analysis, Park-Ang index are considered as a dynamic damage index and the relationship among static damage indices with Park-Ang index has been investigated undergone to an ensemble of 20 strong ground motions. Therefore, the performance point of ZBF structure based on ATC-40 (1997) has been determined and the value of static damage indices in this performance level has been obtained. Finally, the correlation between the Park-Ang index and static damage indices is evaluated by means of some polynomial curves. The results show that this method can be used effectively in determining the amount of structural damages in ZBF structures by using pushover analysis without performing time consuming nonlinear dynamic analyses.

MATERIALS AND METHODS

Damage Indices

Damage on structures is associated with

non-linear behavior, and, hence, the destructive potential of the earthquake must be estimated through the parameters of non-linear structural response. To parametrically investigate structural damage index in ZBF systems a series of well-known, frequently used damage indices already proposed by researchers have been selected. They are considered here such that the effect of different parameters, that could be directly or indirectly associated to the structural damage under earthquake excitations, on damage index of ZBF systems are investigated. Note that all of the selected indices are in the category of the global damage index. Some of the aforementioned indices are non-cumulative index such as plastic ductility and roof drift index, and some are combined indices such as those suggested by Park and Ang (1985) and Bozorgnia and Bertero (2001a,b, 2002) while others are corresponded to modal parameter such as stiffness and maximum softening damage index. The most important damage indices utilized in this study are as follow:

Park- Ang Index

The Park-Ang (1985) damage index, introduced in 1985 for the first time, considers the effects of both parameters of maximum deformation and dissipated energy in damage evaluation. The equation of this index can be expressed as follows:

$$DI = \frac{\Delta_m}{\Delta_u} + \frac{\beta}{V_y.\Delta_u}\int dE_h \quad (1)$$

where Δ_m: is the maximum deformation, Δ_u: is the ultimate deformation, β: is a constant parameter of the model, $\int dE_h$: is the yielding energy and V_y: is yield strength of the element defined as:

$$\Delta_u = \mu.\Delta_y \quad (2)$$

where μ: is the ductility capacity and Δ_y: is the yield displacement. Kunnath et al. (1992) modified the original index to the Eq. (3). While this index has been calibrated for concrete members, it is also usable for damage evaluation of both concrete and steel structures due to its clear physical concepts. The index is popular and is one of the most well-known indices.

$$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y.\theta_u}\int dE_h \quad (3)$$

where θ_m: is the maximum rotation obtained during the loading history; θ_u: is the ultimate rotation capacity of the section; θ_r: is the recoverable rotation during unloading; M_y: is the yield moment and $\int dE_h$: is the dissipated energy in the section. This index is a local damage index and to calculate the global damage index, Park and Ang (1985) presented total damage of a building as an average of local damages weighted by the local energy absorption. Ghosh et al. (2011) proposed the modified Park–Ang damage index for the MDOF model to obtain the global damage index of structures as follows:

$$DI = \frac{\Delta_m - \Delta_y}{\Delta_u - \Delta_y} + \frac{\beta}{V_y.\Delta_u}\int dE_h \quad (4)$$

where Δ_m: is the maximum roof displacement resulted from inelastic response history analysis for a given ground motion; Δ_u: is ultimate monotonic roof displacement; Δ_y: is yield displacement computed from non-linear static pushover analysis; V_y: is yield base shear obtained from pushover analysis and $\int dE_h$: is the dissipated energy of structure. On the basis of the recommendation of Park et al. (1987), the factor β for steel structures is considered as 0.025. Figure 1 shows the Force-displacement relationship under monotonically increasing deformation. In this paper, the modified Park-Ang damage index by Ghosh et al. (2011) is used to obtain the damage index of selected ZBF systems.

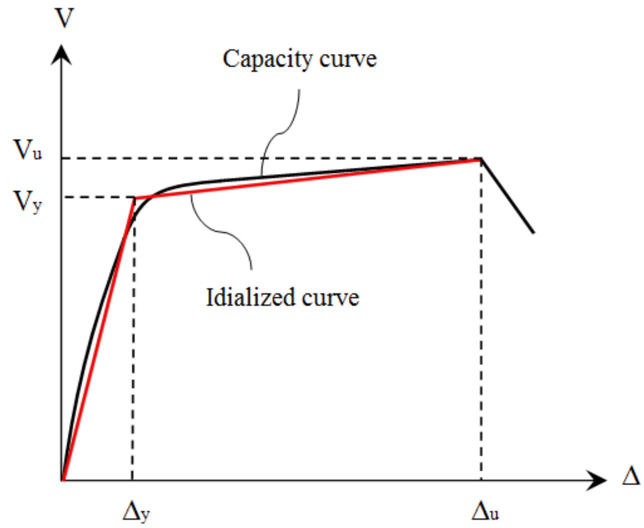


Fig. 1. Pushover curve under monotonically increasing deformation

The dependence of damage degree of the structure from damage index was initiated by Park-Ang (1985). On the basis of data on damage in RC buildings that were moderately or severely damaged during several earthquakes in USA and Japan, they defined the relationship between degree of damage and damage index (Table 1).

Plastic Ductility Index

A damage index corresponding to plastic deformation under monotonically increasing lateral deformation was developed by Powell and Allahabadi (1988), which is defined by Eq. (5). This index is defined as a local damage index and some methods, such as weighted averaging or utilizing the peak value of element indices as the story index, which can be used to globalize it. The simple concept and the practical application make this index a well-known one for practical engineers and researchers.

$$DI = \frac{U_m - U_y}{U_u - U_y} \tag{5}$$

where U_m: is the maximum inelastic displacement during a ground motion, U_y: is the yield displacement and U_u: is an ultimate displacement capacity of the system under a monotonically increasing lateral deformation. In this study, this index is considered as a global damage index as:

$$DI = \frac{\Delta_m - \Delta_y}{\Delta_u - \Delta_y} \tag{6}$$

where Δ_m: is the maximum roof displacement during a ground motion, Δ_y: is the yield displacement and Δ_u: is an ultimate displacement capacity of the structure under a monotonically increasing lateral deformation as illustrated in Figure 1.

Table 1. Park-Ang damage classification levels

Degree of Damage	Damage Index	State of Structure	Performance Level
No Damage	< 0.1	Serviceable	OP
Minor	0.1 – 0.25	Serviceable	IO
Moderate	0.25 – 0.4	Repairable	LS
Severe	0.4 – 1.0	Irreparable	CP
Collapse	> 1.0	Loss of story or buildings	C

Roof Drift Index

One of the most practical damage indices among engineers is drift ratio which is classified as global damage index. This index is also recommended by existing seismic guidelines such as FEMA-273 (1997) and ATC-40 (1997) for evaluation of the performance level of the structure:

$$DI = \frac{\Delta_m}{H} \tag{7}$$

where Δ_m: is the maximum roof displacement during a ground motion and H: is total height of the structure.

Stiffness Index

Ghobarah et al. (1999) proposed a simple global damage index based on structural stiffness formulated with the Eq. (7). This approach is to perform pushover analysis for the structure twice; once before and once after subjecting the structure to the earthquake ground motion.

$$DI = 1 - \frac{K_{final}}{K_{initial}} \tag{8}$$

where $K_{initial}$: is initial stiffness or the starting tangent of the base shear-roof displacement curve before the earthquake event and K_{final}: is the tangent of the curve after the earthquake event. The amount of DI ranges from 0 to 1.0; 0 represents no damages and 1.0 represents incipient collapse of the structure.

Maximum Softening Index

Dipasquale and Cakmak (1990) defined the maximum softening damage index as Eq. 9, where only the fundamental design frequency is considered. This definition has advantage of yielding and the value of maximum softening that is always between 0 and 1, as it is customary for damage indices.

$$DI = 1 - \frac{T_{initial}}{T_{\max}} \tag{9}$$

where $T_{initial}$: is period of undamaged structure and T_{max}: is maximum period of structure.

Bozorgnia and Bertero Index

Bozorgnia and Bertero (2001a,b, 2002) proposed two modified damage indices for an equivalent inelastic SDOF system. These damage indices are regarded as follows:

$$DI_1 = \frac{(1-\alpha_1)(\mu-\mu_e)}{\mu_{mon}-1} + \alpha_1 \frac{E_H}{E_{Hmon}} \tag{10}$$

$$DI_2 = \frac{(1-\alpha_2)(\mu-\mu_e)}{\mu_{mon}-1} + \alpha_2 \left(\frac{E_H}{E_{Hmon}}\right)^{0.5} \tag{11}$$

where,

$$\mu = \frac{\mu_{\max}}{\mu_y} \tag{12}$$

$$\mu_e = \frac{\mu_{elastic}}{\mu_y} = \begin{cases} 1 & for\ inelastic\ behavior \\ \mu & if\ the\ response\ remains\ elastic \end{cases} \tag{13}$$

μ_{mon}: is monotonic displacement ductility capacity, E_H is hysteretic energy demanded by the earthquake ground motion, E_{Hmon}: is hysteretic energy capacity under monotonically increasing lateral deformation, and $0 \le \alpha_1 \le 1$ and $0 \le \alpha_2 \le 1$ are constants.

Conception of Performance Level and Performance Point

There is different performance levels defined in the FEMA-356 (2000):

1. Immediate Occupancy (IO): The structural elements are partially damaged.
2. Life Safety (LS): The structural and non-structural elements are remarkably damaged.

3. Collapse Prevention (CP): The structure is about to collapse.
4. Collapse (C): The structure fully collapsed.

The performance level of a structure is assessed by evaluating two damage variables: Drift and Plastic deformation. In the current paper the drift criteria has been used to quantify the performance levels of ZBF structures. The performance levels of steel braced frames in FEMA-356 (2000) based on story drift are shown in Table 2.

To calculate the damage index by using pushover analysis, first the performance points of the structures is determined based on ATC-40 (1997) capacity spectrum method (CSM) and then the values of the damage indices are computed in the performance points. The capacity spectrum method (CSM) is a nonlinear static procedure that exhibits a graphical representation of the global force-deformation capacity curve of the structure (i.e., pushover) and is a very useful tool in the determination of performance point of buildings. To do this, both the capacity curve and the response spectra need to be converted into a spectral acceleration S_a spectral displacement S_d graphs. By using a trial and error procedure one can estimate the performance point of a structure. In fact, a performance point for a structure can be calculated for each level of intensity by means of capacity spectrum method (CSM). Figure 2 shows the performance point in capacity spectrum method.

Modeling and Assumptions

Zipper-braced frames are one of the innovative load-resisting systems firstly introduced by Khatib et al. (1988), and developed by other researchers during the last decade (Khatib et al., 1988; Sabelli, 2001; Tremblay and Trica, 2003; Yang et al., 2008). Khatib et al., (1988) proposed to link all beam-to-brace inter-section points of adjacent floors and to transfer the unbalanced load to the vertical member called “zipper column” and this new structural system is called “zipper braced frame”. The main application of zipper braced frame (ZBF) is to tie all brace-to-beam intersection points together, and force all compression braces in a braced bay to buckle simultaneously. In the last study, a suspended ZBF system was developed by Yang et al. (2008).

Table 2. FEMA-356 performance levels

Performance Level	IO	LS	CP	C
Story Drift (%)	0.5	1.5	2	> 2

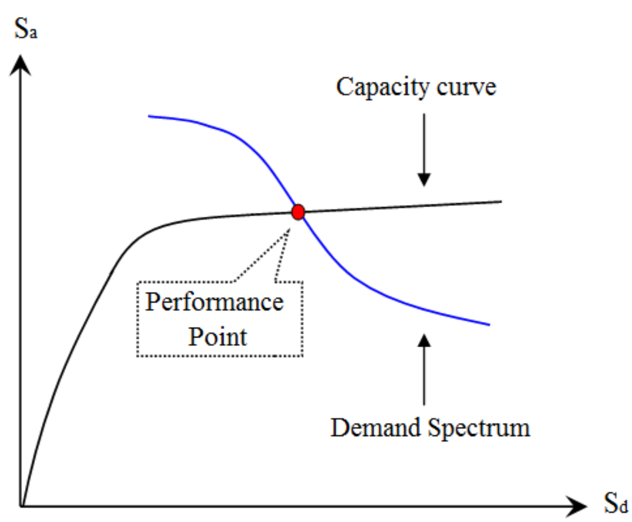


Fig. 2. Determination of performance point in capacity spectrum method

This new structure consists of adding an elastic truss at the roof floor level in which braces were designed to behave elastically to avoid the full-height zipper mechanism formation. All the remaining braces were proportioned to buckle and zippers to yield. In fact, they developed a new design approach and configuration of Zipper Braced System called suspended zipper braced frame (S-ZBF). Recently, Vaseghi et al. (2015) proposed a new method to access the minimum seismic damage for ZBF structures by using stories ductility ratio as damage index criteria. In another study they also comprehensively investigated the ductility reduction factors for zipper-braced frames under strong ground motion excitation (Vaseghi et al., 2016). They considered 1, 5, 10 and 15-story models including zipper-braced frames (ZBF), MDOF shear buildings and SDOF systems to represent a wide range of building structures. More than 1,000,000 nonlinear dynamic analyses were performed under twenty different synthetic seismic ground motions and the ductility-dependent reduction factors of the models were computed. Based on the results of conducted study, a simple equation was proposed to calculate the ductility reduction factor of zipper-braced frames.

In this paper, In order to evaluate parametrically the performance levels of various ZBF structures using several structural damage indices, instead of utilizing more precise 3D models, it is inevitable to consider regular 2D models. For this purpose, five zipper braced frames (ZBFs) with 5, 7, 10, 12 and 15 stories are seismically loaded based on ASCE7-10 (2010) lateral load pattern and designed based on AISC-LRFD (2005). It is supposed that all the models are regular in plan and height and the story height and span length of all models are 3 and 6 meters, respectively.

The general pattern of the seismic load pattern specified by the ASCE7-10 (2010) is defined as:

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{14}$$

where F_x and V: are respectively the lateral load at level x and the design base shear; w_i and w_x: are the portion of the total gravity load of the structure located at the level i or x; h_i and h_x: are the height from the base to the level i or x; n: is the number of stories; and k: is an exponent that differs from one seismic code to another. In ASCE7-10 (2010), k: is related to the fundamental period of the structure. Figure 3 shows typical ZBF models that used in this study. The model specification and the member's characteristics are presented in Tables 3 and 4.

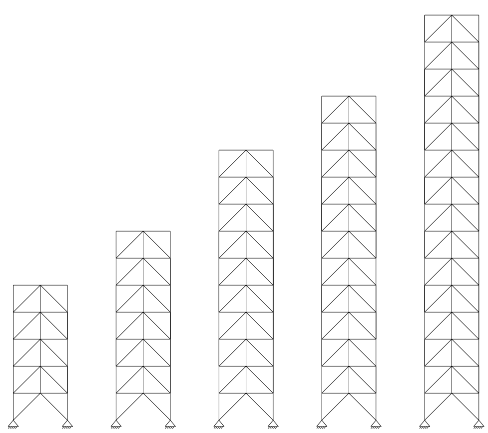


Fig. 3. Typical ZBF models

Table 3. ZBF model properties

Number of Story	**5**	**7**	**10**	**12**	**15**
Total height (m)	15	21	30	36	45
Natural period (sec)	0.33	0.41	0.62	0.72	0.92

Table 4. The member's characteristics of ZBF models

Model	**Story**	**Beam**	**Column**	**Zipper Column**	**Braces**
5-story	5	IPE 180	IPB 100	IPB 220	Box 140x140x10
	4	IPE 220	IPB 100	IPB 180	Box 80x80x5
	3	IPE 220	IPB 160	IPB 140	Box 80x80x5
	2	IPE 220	IPB 180	IPB 100	Box 80x80x5
	1	IPE 220	IPB 220	-	Box 80x80x5
7-story	7	IPE 180	IPB 100	IPB 300	Box 180x180x12.5
	6	IPE 220	IPB 100	IPB 280	Box 80x80x5
	5	IPE 220	IPB 160	IPB 240	Box 80x80x5
	4	IPE 240	IPB 200	IPB 200	Box 80x80x5
	3	IPE 240	IPB 240	IPB 160	Box 80x80x7.1
	2	IPE 240	IPB 280	IPB 100	Box 80x80x7.1
	1	IPE 240	IPB 320	-	Box 80x80x7.1
10-story	10	IPE 180	IPB 100	IPB 450	Box 260x260x16
	9	IPE 220	IPB 100	IPB 400	Box 80x80x5
	8	IPE 220	IPB 160	IPB 360	Box 80x80x5
	7	IPE 240	IPB 200	IPB 320	Box 80x80x5.9
	6	IPE 240	IPB 240	IPB 280	Box 90x90x5
	5	IPE 240	IPB 280	IPB 240	Box 90x90x5
	4	IPE 240	IPB 300	IPB 220	Box 90x90x5
	3	IPE 240	IPB 360	IPB 180	Box 90x90x7.1
	2	IPE 240	IPB 450	IPB 120	Box 90x90x7.1
	1	IPE 240	IPB 500	-	Box 90x90x7.1
12-story	12	IPE 180	IPB 100	IPB 650	Box 300x300x16
	11	IPE 200	IPB 100	IPB 600	Box 80x80x5
	10	IPE 240	IPB 160	IPB 500	Box 80x80x7.1
	9	IPE 240	IPB 200	IPB 450	Box 80x80x7.1
	8	IPE 240	IPB 240	IPB 400	Box 90x90x5
	7	IPE 240	IPB 280	IPB 340	Box 90x90x5
	6	IPE 240	IPB 320	IPB 300	Box 90x90x7.1
	5	IPE 240	IPB 400	IPB 260	Box 90x90x7.1
	4	IPE 240	IPB 450	IPB 220	Box 90x90x7.1
	3	IPE 240	IPB 550	IPB 180	Box 90x90x7.1
	2	IPE 240	IPB 650	IPB 120	Box 90x90x7.1
	1	IPE 240	IPB 800	-	Box 90x90x7.1
15-story	15	IPE 180	IPB 100	IPB 1000	Box 300x300x12.5
	14	IPE 220	IPB 100	IPB 900	Box 80x80x5
	13	IPE 240	IPB 160	IPB 900	Box 80x80x6.3
	12	IPE 240	IPB 200	IPB 800	Box 80x80x6.3
	11	IPE 240	IPB 240	IPB 700	Box 90x90x5
	10	IPE 240	IPB 280	IPB 600	Box 90x90x6.3
	9	IPE 240	IPB 320	IPB 500	Box 90x90x6.3
	8	IPE 240	IPB 400	IPB 450	Box 90x90x6.3
	7	IPE 240	IPB 500	IPB 360	Box 100x100x6.3
	6	IPE 240	IPB 600	IPB 300	Box 100x100x6.3
	5	IPE 240	IPB 700	IPB 260	Box 100x100x6.3
	4	IPE 240	IPB 800	IPB 220	Box 100x100x6.3
	3	IPE 240	IPB 900	IPB 180	Box 100x100x6.3
	2	IPE 240	IPB 1000	IPB 120	Box 100x100x6.3
	1	IPE 240	IPB 1000	-	Box 100x100x6.3

All the nonlinear static and dynamic analyses were conducted by OPENSEES (Mazzoni et al., 2016). It allows the users to create structural Finite Element models and numerical applications for simulation form the response analysis of the structural and geotechnical systems subjected to earthquakes. To model the buckling behavior of the ZBF models a uniaxial material interface to define the brace's force-deformation relationship of a brace has been utilized that is called "steel 01". Therefore, a bilinear elasto-plastic model with 3% strain hardening has been used to represent the Uniaxial Material "steel01" force-deformation relationship. In addition, to simulate the buckling behavior of a brace under compression for the hysteretic response of the zipper frame model a brace model with a small initial imperfection has been defined (Uriz and Mahin, 2004). Figures 4a and 4b show the Uniaxial Material "steel01" force-deformation relationship and schematic graph of a brace model in ZBF structures, respectively. F_y, E and α are the yield strength, modules of elasticity and strength hardening ratio of Uniaxial Material steel01, respectively.

Earthquake Records

To evaluate the amount of damage indices and performance levels of selected structures, a family of twenty strong ground motions is utilized. They are obtained from the Earthquake strong ground motion with various characteristics recorded on a very dense soil of type D according to the IBC-2012 (2012). The selected ground motions are components of ten earthquake events including Imperial Valley 1979, Morgan Hill 1984, Kocaeli, 1999, Loma Prieta 1989, Northridge 1994, Landers 1992, N. Palm Springs 1986, Victoria 1980, Borrego Mtn 1968 and Whittier Narrows 1987. The main properties of the ground motions are provided in Table 5. All the ground motions have magnitude larger than 6 with closest distance to fault rupture greater than 15 km. To be consistent, using SeismoMatch (2016) software the selected seismic ground motions are adjusted to the elastic design response spectrum of IBC-2012 (2012) with soil type *C*. Figure 5shows a comparison of the 20 matched ground motion spectra with the target elastic design response spectrum of IBC-2012 (2012). In order to determine the correlation between structural damage indices and FEMA-356 (2000) drift criteria in a wide range of earthquake ground motion intensities, the response spectrum of IBC-2012 shown in Figure 5 is scaled by factors 1.5, 2, 2.5 and 3. Finally, the nonlinear dynamic analyses have been conducted by OPENSEES (Mazzoni et al., 2016).

Table 5. Selected ground motions soil type C on the basis of USGS site classification

Earthquake	Year	Station	Component	Distance	Soil	PGA (g)
Borrego Mtn	1968	117 El Centro Array #9	270,15	46	C	0.130,0.057
Imperial Valley	1979	6622 Compuertas	285,15	32.6	C	0.186,0.147
Kocaeli	1999	Iznik	180,90	31.8	C	0.098,0.136
Landers	1992	12025 Palm Springs	0,90	37.5	C	0.076,0.089
Loma Prieta	1989	47179 Salinas	160,250	32.6	C	0.091,0.112
Morgan Hill	1984	1028 Hollister City Hall	1,271	32.5	C	0.071,0.071
N. Palm Springs	1986	12331 Hemet Fire Station	270,360	43.3	C	0.144,0.132
Northridge	1994	25282 Camarillo	180,270	36.5	C	0.125,0.121
Victoria	1980	6621 Chihuahua	102,192	36.6	C	0.150,0.092
Whittier Narrows	1987	90003 Northridge	90,180	39.8	C	0.161,0.118

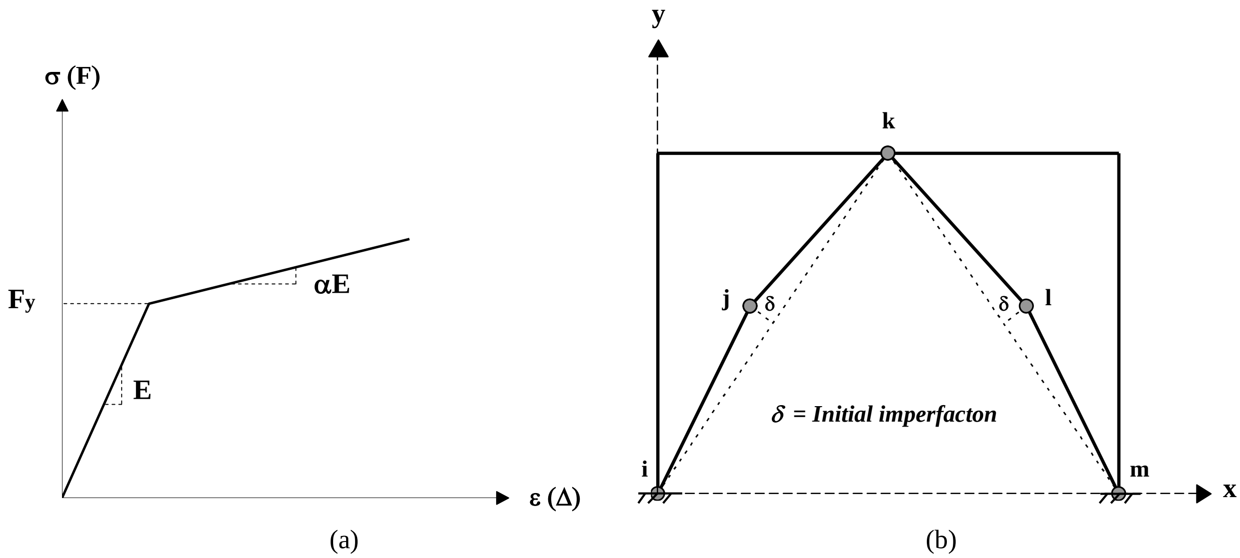


Fig. 4. a) Characteristics of Uniaxial Material, steel01, b) Schematic graph of a brace model in ZBF structures

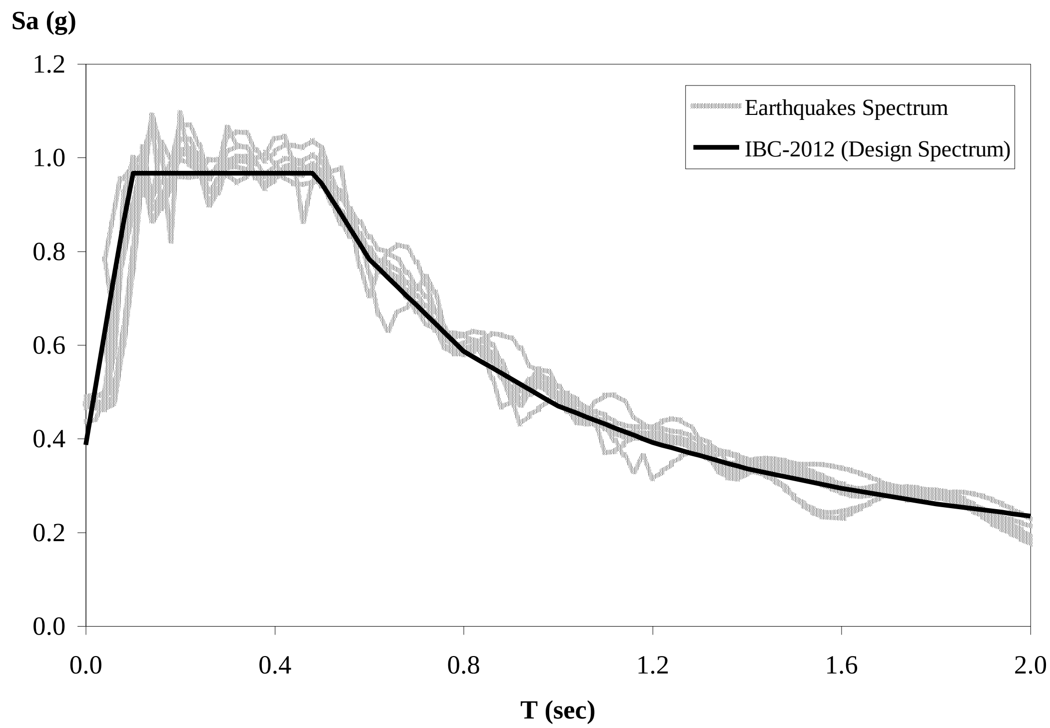
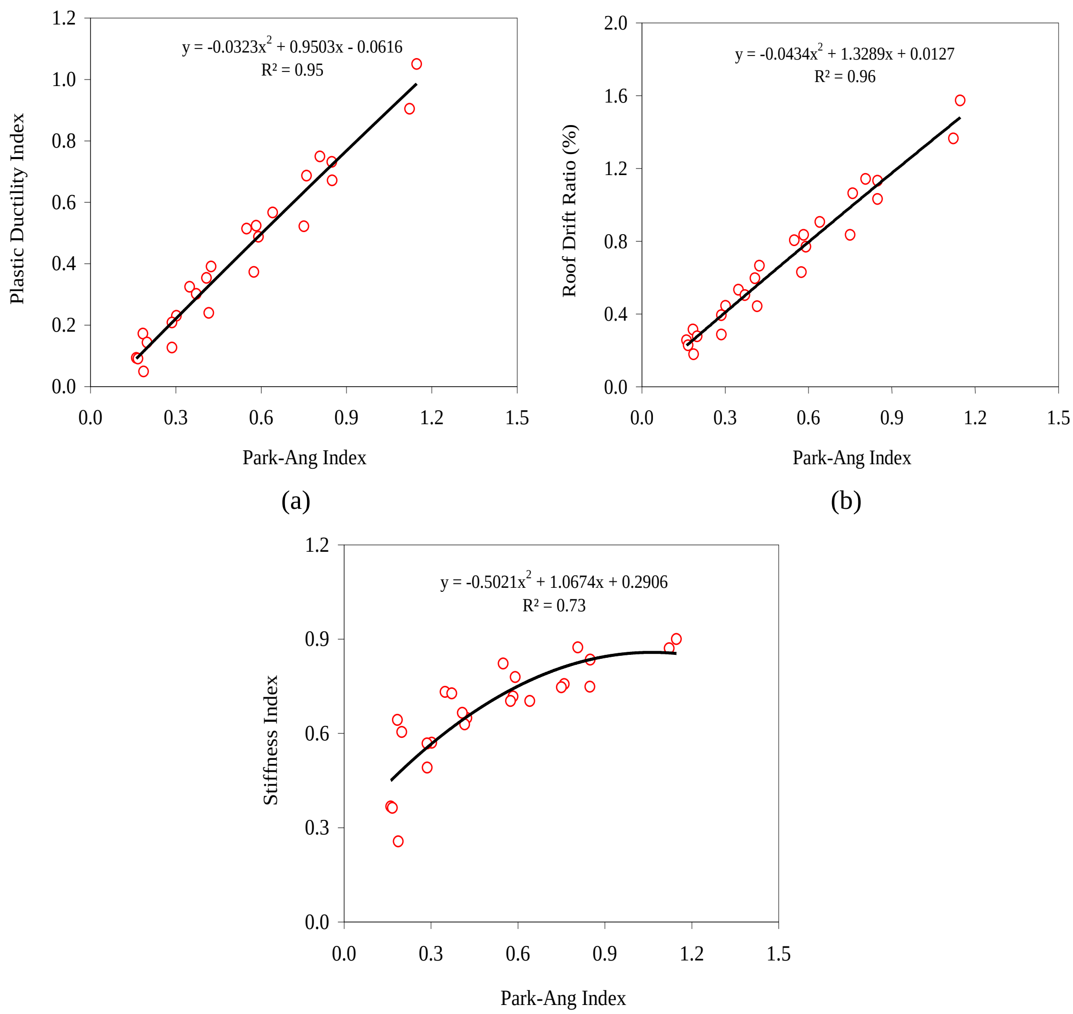


Fig. 5. IBC-2012 (2012) design spectrum for soil type C and response spectra of 20 earthquakes (5% damping) for selected ground motions

RESULS AND DISCUSSIONS

Calculation of ZBF Performance Levels by Using Pushover Analysis

In this section, the correlation between Park-Ang index as a dynamic damage index, and plastic ductility, stiffness and roof drift indices as static damage indices is investigated. For this Purpose, thousands of dynamic nonlinear analyses and pushover analyses have been conducted on ZBF structures and the value of these damage indices has been calculated. To determine the relation between static and dynamic damage indices in a wide range of the damages values, five performance levels were considered for each frame. These levels correspond to 1, 1.5, 2, 2.5 and 3 times the design response spectrum of IBC-2012 (2012) already explained in the previous section and the value of the static and dynamic damage indices were calculated. It means that by using the pushover analysis the performance points of ZBF models based on ATC-40 (1997) capacity spectrum method (CSM) are obtained and consequently the static damage indices are calculated at theses performance levels. Then, by doing the nonlinear dynamic analyses on ZBF structures the dynamic damage index is determined subjected to the 20 earthquake ground motions matched to predefined spectrums. Finally, the correlation between dynamic damage index and each of static damage indices is evaluated. In the current paper, the correlations between Park-Ang damage index and roof drift, plastic ductility and stiffness damage indices are evaluated and the results provided in Figure 6. In these Figures, each point represents the average of structural response under 20 earthquake ground motions for dynamic damage index and the performance level of structure for static damage index.



(c)

Fig. 6. Correlation between static damage indices and Park-Ang damage index; a) Plastic Ductility index, b) Roof Drift index, c) Stiffness index

As seen in Figure 6, the correlation of dynamic and static damage indices has been evaluated by applying the second order curve. The results show that there is a good correlation between them. The correlation among the selected damage indices is considered to be satisfactory as the values of correlation coefficient R^2 for drift, plastic ductility and stiffness damage indices and Park-Ang damage index are 0.96, 0.95 and 0.73 respectively. The numerical range of the aforementioned damage index corresponding to ZBF performance levels can be easily obtained using the above graphs and the values of Table1. Therefore, using the numerical data presented in Figure 6, the value of each damage index associated to ZBF performance levels can be presented in Table 6.

The performance levels of ZBF structures are developed based on Park-Ang damage classification levels that introduced in Table1. The results of Table 6 can be used effectively to determine the value of damage and performance levels of ZBF structures based on the results of pushover analysis without performing complicated nonlinear dynamic analyses. The roof drift index results provided in Table 6 show that the performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) for ZBF structures are initiated at drift ratio of 0.15, 0.34 and 0.54, respectively. Comparing the results of drift index with those of FEMA-356 (2000) drift criteria as presented in Table 2 indicate that the proposed FEMA-356 (2000) drift criteria should be revised for ZBF structures.

Calculation of ZBF Performance Levels by Using Nonlinear Dynamic Analysis

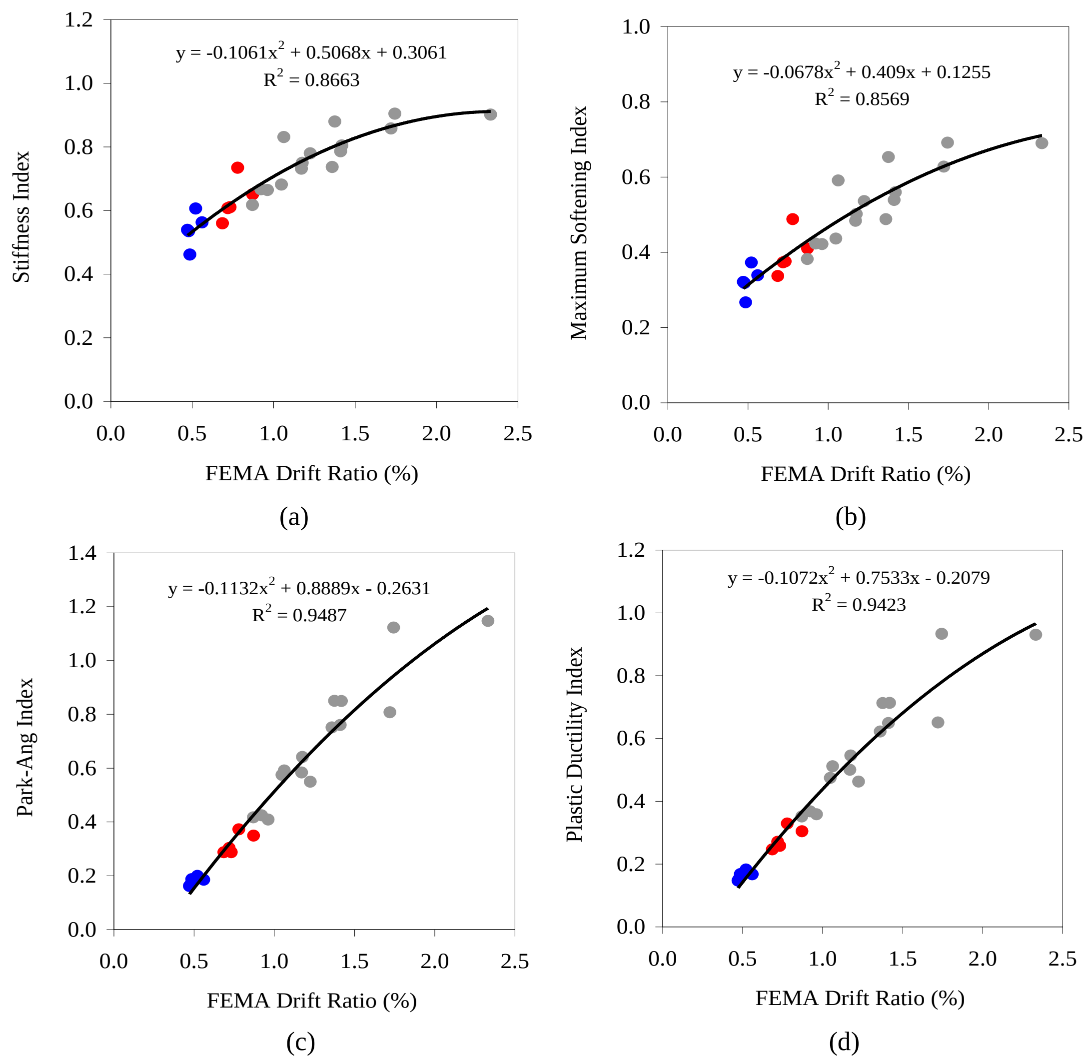
In this section, the values of damage indices by using nonlinear dynamic analyses subjected to earthquake excitations are calculated. Then, the correlation between FEMA-356 (2000) performance levels and damage indices introduced in the previous section is investigated. To do this, numerous nonlinear static and dynamic analyses have been performed on realistic ZBF models. Then, the values of damage indices and maximum inter-story drift of each structure subjected to the 20 earthquake ground motions have been calculated based on FEMA-356 (2000) damage criteria and the results are presented in Figure 7. Each data point represents the average of structural response under 20 spectrum-compatible earthquakes. This figure shows the correlation between FEMA-356 (2000) and respectively the damage indices of drift criteria with Park-Ang, Bozorgnia and Bertero, plastic ductility, roof drift and, maximum softening and as well as stiffness damage indices.

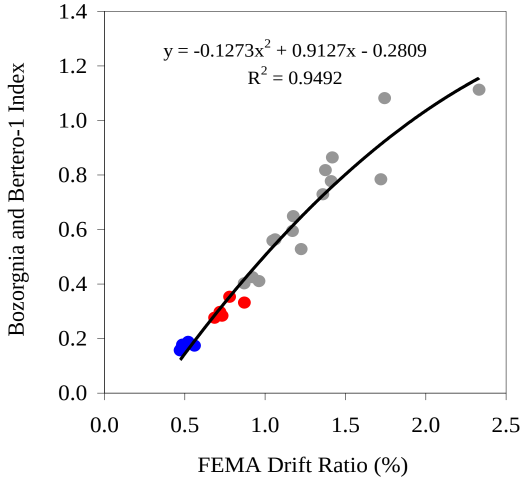
In the presented figures, the blue points correspond to the damage indices values under IBC-2012 spectrum-compatible design earthquakes. The design earthquake ground motions (D.E) are defined as ground shaking having a 10% probability of exceedance in 50 years and can be considered as Basic Safety Earthquake-1 (BSE-1) hazard level in FEMA-356 (2000). Also the red points correspond to the damage indices values under Maximum Considered Earthquake (M.C.E) defined as an extreme earthquake hazard level by MCE maps. This seismic hazard map value indicate ground motions that have a probability of being exceeded in 50 years of 2 percent, and is equal to Basic Safety Earthquake-2 (BSE-2) hazard level in FEMA-356 (2000). According to FEMA-356 (2000) guidelines, the site-specific response acceleration parameters for the BSE-2 earthquake hazard level can be obtained as the values of the parameters from 150% of median deterministic site-specific spectra (IBC-2012 design spectrum). Therefore, the damage indices values correspond to earthquakes adjusted to 1.5 times the design spectrum can be adopted as M.C.E (BSE-2) earthquakes responses. The gray points also correspond to 2, 2.5 and 3 scale factors.

As shown in Figure 7, the correlation between damage indices and FEMA-356 (2000) damage criteria can be estimated by a second order curve. As observed, there are good correlation between FEMA-356 (2000) damage criteria and damage indices. The correlation coefficient R^2 for the damage indices ranges from 0.85 to 0.95. The highest correlation coefficient (0.95) is for the case of Bozorgnia and Bertero-2 damage index. Also, the stiffness and maximum softening indices were somewhat scattered with respect to FEMA-356 (2000) damage criteria. The numerical range of each damage index correlated to FEMA-356 performance levels can be derived using the above graphs. As a result, using the equations obtained above, the value of each damage index correlated to FEMA-356 (2000) performance levels can be presented in Table 7.

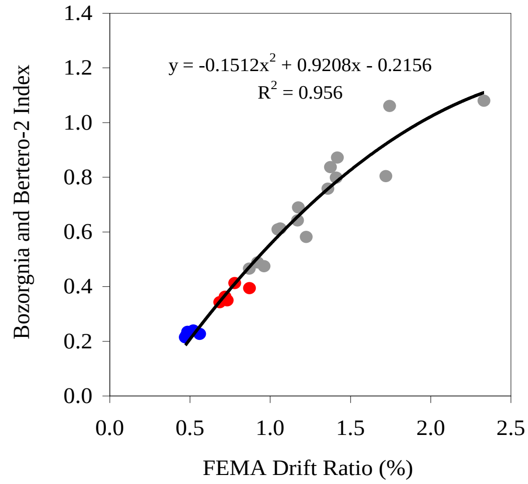
Table 6. The value of damage indices associated to performance levels of ZBF structures

Damage Index	ZBF Performance Levels		
	IO	**LS**	**CP**
Roof Drift (%)	0.15 - 0.34	0.34 - 0.54	0.54 - 1.30
Plastic Ductility	0.03 - 0.17	0.17 - 0.31	0.31 - 0.86
Stiffness	0.39 - 0.53	0.53 - 0.64	0.64 - 0.86

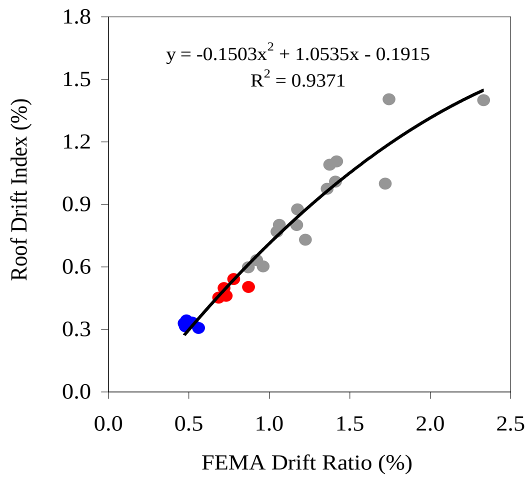




(e)



(f)



(g)

Fig. 7. Correlation between damage indices and FEMA drift ratio; a) Stiffness index, b) Maximum softening index, c) Park-Ang index, d) Plastic Ductility index, e) Bozorgnia and Bertero-1 index, f) Bozorgnia and Bertero-2 index, g) Roof Drift index

Table 7. The value of damage indices correlated to FEMA-356 performance levels

Damage Index	**FEMA-356 Performance Level**		
	IO	**LS**	**CP**
Park-Ang	0.0 - 0.15	0.15 - 0.82	0.82 - 1.0
Maximum Softening	0.0 - 0.31	0.31 - 0.59	0.59 - 0.67
Stiffness	0.0 - 0.53	0.53 - 0.83	0.83 - 0.90
Plastic Ductility	0.0 - 0.14	0.14 - 0.68	0.68 - 0.87
Roof Drift (%)	0.0 - 0.30	0.30 - 1.05	1.05 - 1.31
Bozorgnia and Bertero-1	0.0 - 0.14	0.14 - 0.80	0.80 - 1.0
Bozorgnia and Bertero-2	0.0 - 0.21	0.21 - 0.83	0.83 - 1.0

From Table 7, it can be observed that the relationship between Park-Ang damage index and performance levels of ZBF structures is compatible with the Park-Ang proposed values (Table 1) for *IO* and *CP* performance levels and incompatible for *LS* performance level. Because the values of Table 1 was calibrated based on the RC frames data and they may not accurate for ZBF structures. To estimate the Performance level of ZBF structures for Basic Safety Earthquake-1 (BSE-1) and Basic Safety Earthquake-2 (BSE-2) hazard levels, the value of damage indices with blue and red points specified in Figure 7 are selected, and the maximum values of damage indices and performance levels of ZBF models based on Table 7 for aforementioned hazard levels are presented in Table 8.

The results illustrated in Table 8 show that the Performance level of ZBF structures in BSE-1 and BSE-2 hazard level is *LS*. It shows the good performance of seismic design requirements in provisions to design the ZBF structures and limitation of damages in these systems. Also, the results of Yang et al. (2008) studies on the performance of the ZBF models using nonlinear dynamic analyses under an ensemble of 2%-in-50-year pulse-type near-fault ground motions confirmed the results of this study. Yang et al. (2008) analyses indicated that the design procedure produces safe designs in ZBF structures and satisfies inter-story drifts limitation.

Overall, to show the analysis process through a flowchart for determination of ZBF performance level based on nonlinear static (pushover) and nonlinear dynamic analyses Figures 8 and 9 are provided. As can be seen, an iterative analysis must be carried out to obtain the performance level based on the given damage analysis.

CONCLUSIONS

In this paper, a new method was developed to determine the performance levels of ZBF structures based on the results of nonlinear static and dynamic analyses. For this purpose, a large number of nonlinear dynamic analyses were performed under twenty different synthetic seismic ground motions and the damage indices of these models have been calculated. On the basis of nonlinear static damage analysis, by using the pushover analysis of the ZBF structures and determination of damage indices in their performance points, the relationship between dynamic and static damage index has been evaluated. On the other side, based on dynamic damage analysis, the correlation between FEMA-356 (2000) drift index and the value of damage index was evaluated and the results are tabulated for practical purpose. Based to the results of this study, the results of nonlinear static damage analysis are summarized below:

- There is a good correlation between Park-Ang damage index and nonlinear static damage indices as the correlation coefficient R^2 for plastic ductility and drift indices are more than 0.95 but for stiffness index is 0.73.
- The correlation between Park-Ang damage index and plastic ductility and drift indices shows the appropriate performance of these damage indices in damage analysis of ZBF structures by pushover method.
- The results of roof drift index with FEMA-356 drift criteria show that the proposed FEMA-356 drift criteria could be revised for ZBF structures.

Table 8. The relation between damage indices and performance level of ZBF structures for BSE-1 and BSE-2 hazard level

Damage Index	DI		Performance Level	
	BSE-1	BSE-2	BSE-1	BSE-2
Park-Ang	0.20	0.37	LS	LS
Maximum Softening	0.37	0.49	LS	LS
Stiffness	0.61	0.73	LS	LS
Plastic Ductility	0.18	0.33	LS	LS
Roof Drift (%)	0.34	0.54	LS	LS
Bozorgnia and Bertero-1	0.19	0.35	LS	LS
Bozorgnia and Bertero-2	0.24	0.41	LS	LS

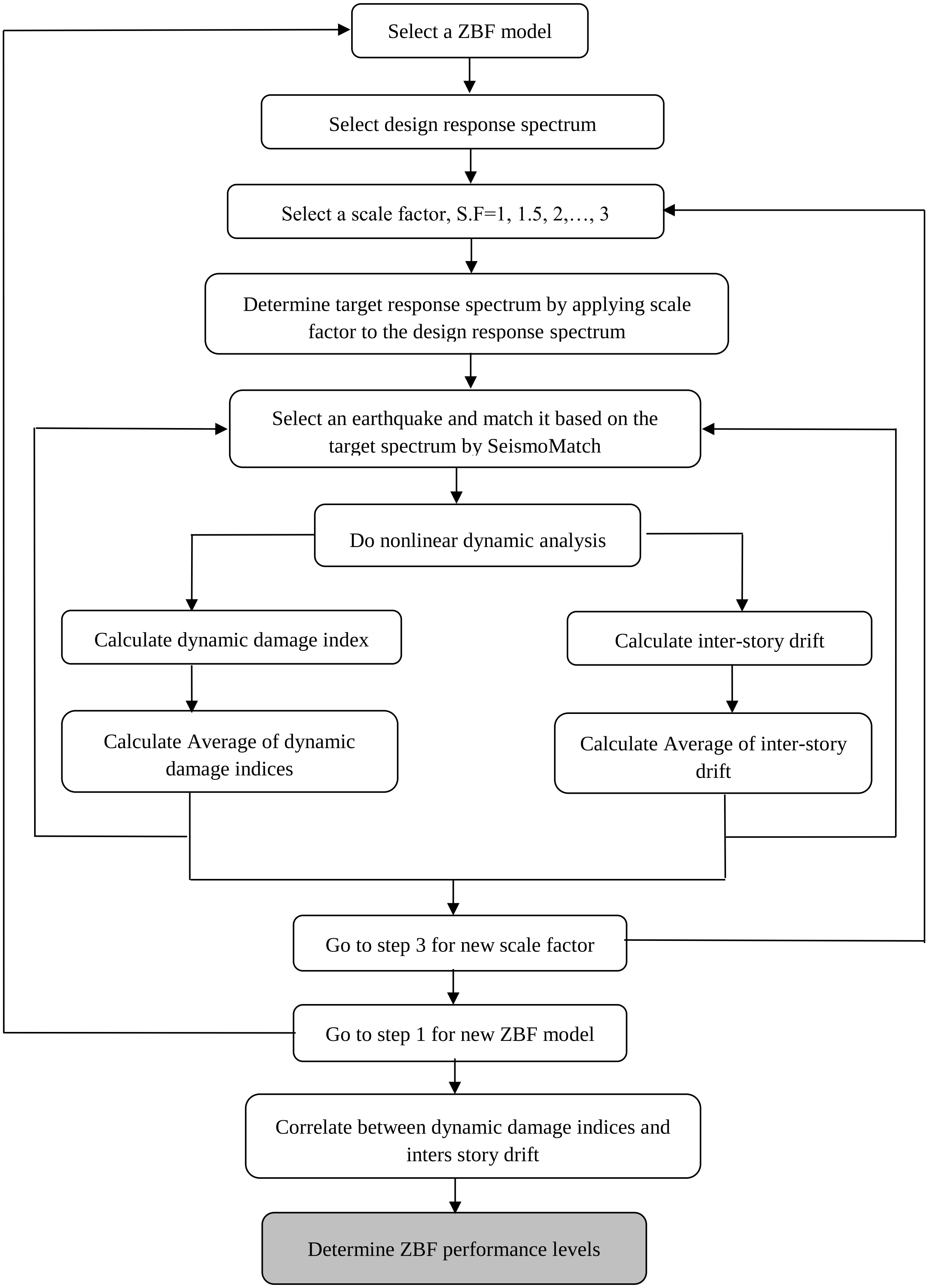


Fig. 8. A flowchart showing the nonlinear static analysis for determination of ZBF performance level

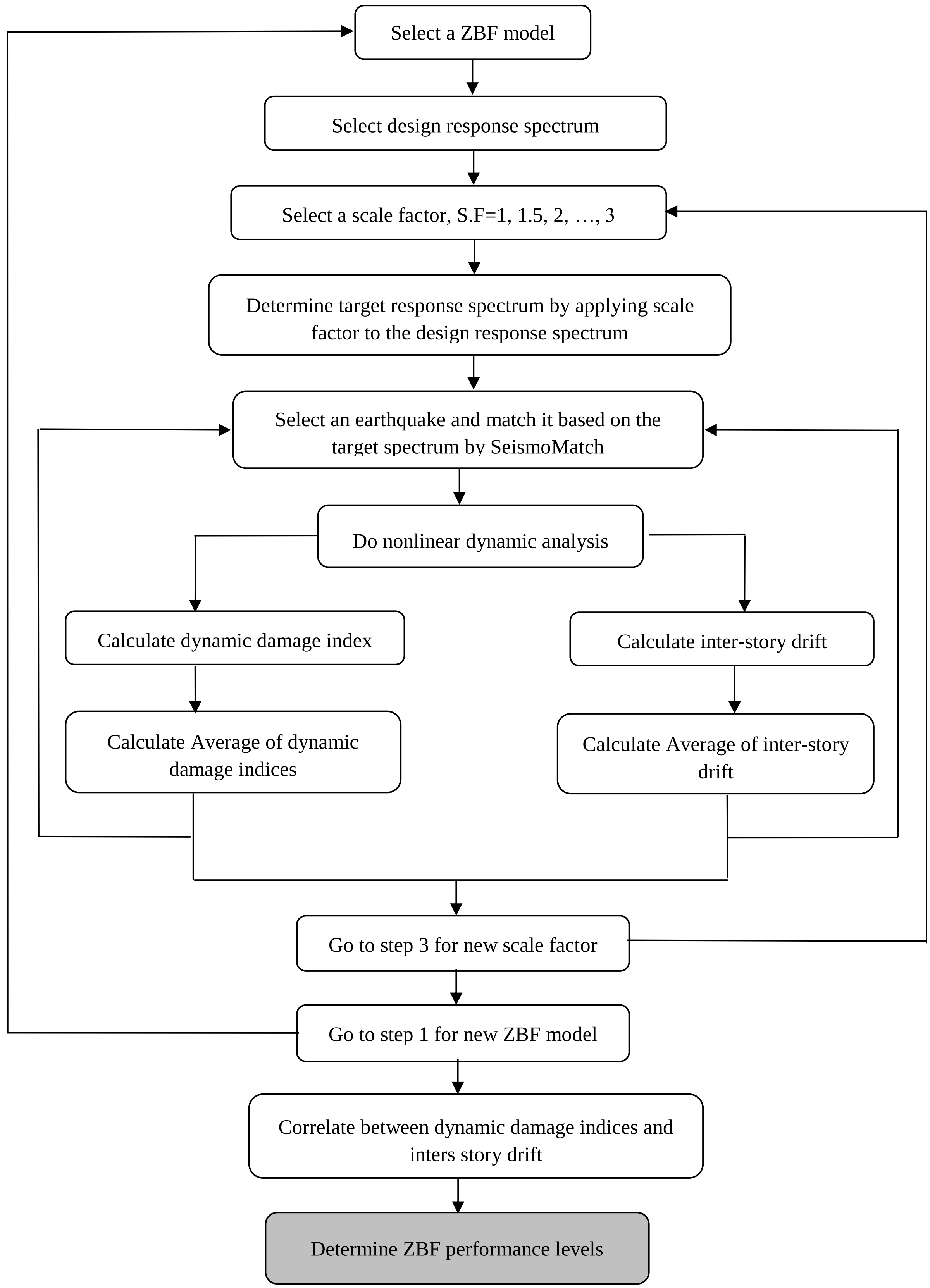


Fig. 9. A flowchart showing the nonlinear dynamic analysis for determination of ZBF performance level

Also the result of dynamic damage analysis can be summarized as:

- There is a good correlation between damage indices and FEMA-356 damage criteria as the correlation coefficients R^2 for combined indices and cumulative damage indices are more than 0.93 while for modal indices are close to 0.85. However, care should be taken into account for when using the values of proposed equation and tables for maximum softening and stiffness indices.
- By comparing the values of Table 7 and without considering the values of stiffness and maximum softening indices because of their scatter responses, it can be said that when the range of damage indices in ZBF structures is 0-0.24, 0.24-0.75 and 0.75-0.9, the performance level of them is *IO*, *LS* and *CP*, respectively.
- According to the new range of damage indices for ZBF structure defined in this study, except for life safety (LS) performance level, there is an acceptable agreement between Park-Ang damage range (Table 1) and the new definition of damage index range.
- The relationship between damage indices and performance level of ZBF structures for BSE-1 and BSE-2 hazard level shows that the ZBF systems have good performance in restricting the structural damages because the performance level of ZBF structures at these hazard levels is limited to *LS*.

REFERENCES

Abdollahzadeh, G., Sazjini, M. and Asghari, A. (2015). "Seismic fragility assessment of Special Truss Moment Frames (STMF) using the capacity spectrum method", *Civil Engineering Infrastructures Journal*, 48(1), 1-8.

AISC-LRFD, (2005). *Seismic provisions for structural steel buildings*, American Institute of Steel Construction, Chicago.

Arjomandi, K., Estekanchi, E. and Vafai, A. (2009). "Correlation between structural performance levels and damage indexes in steel frames subjected to earthquakes", *Scientia Iranica, Transaction A: Civil Engineering*, 16(2), 147-155.

ASCE7-10, (2010). *Minimum design loads for buildings and other structures*, American Society of Civil Engineers: Reston, VA.

ATC-40, (1997). *Seismic evaluation and retrofit of concrete buildings*, California Seismic Safety Commission, Applied Technology Council.

Banon H. and Veneziano, D. (1982). "Seismic safety of reinforced concrete members and structures", *Earthquake Engineering and Structural Dynamics*, 10(2), 179-193.

Bertero, R.D. and Bresler, B. (1971). "Seismic safety of reinforce concrete members and structure", *Earthquake Engineering and Structural dynamic*, 10, 179-193.

Bozorgnia, Y. and Bertero, V.V. (2001a). "Evaluation of damage potential of recorded earthquake ground motion", *96th Annual Meeting of Seismological Society of America*.

Bozorgnia, Y. and Bertero, V.V. (2001b). "Improved shaking and damage parameters for post-earthquake applications", *Proceedings of SMIP01 Seminar on Utilization of Strong-Motion Data*, Los Angeles.

Bozorgnia, Y. and Bertero, V.V. (2002). "Improved damage parameters for post-earthquake applications", *Proceedings of SMIP02 Seminar on Utilization of Strong-Motion Data*, Los Angeles.

Bracci, J.M., Reinhorn, A.M., Mander, J.B. and Kunnath, S.K. (1989). "Deterministic model for seismic damage evaluation of reinforced concrete structures", Technical Report NCEER-89-0033, State University of New York, Buffalo.

Colombo, A. and Negro, P.A. (2005). "Damage index of generalized applicability", *Engineering Structures*, 27(8), 1164-1174.

DiPasquale, E. and Cakmak, A.S. (1990). "Seismic damage assessment using linear models", *Soil Dynamics and Earthquake Engineering*, 9(4), 194-215.

Elenas, A. (2013). "Intensity parameters as damage potential descriptors of earthquakes", *in Computational Methods in Stochastic Dynamics*, Springer, 327-334.

Fardis, M.N. (1994). "Damage measures and failure criteria for reinforced concrete members", *Proceedings of 10th European Conference on Earthquake Engineering*, Balkema (Rotterdam), Vienna.

FEMA-273, (1997). *NEHRP guidelines for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, Washington D.C.

FEMA-356, (2000). *Standard and commentary for the* *seismic rehabilitation of buildings*, Federal Emergency Management Agency.

Ghobarah, A., Abou-Elfath, H. and Biddah, A. (1999). "Response-based damage assessment of structures", *Earthquake Engineering and Structural Dynamics*, 28(1), 79-104.

Ghosh, S., Datta, D. and Katakdhond, A.A. (2011). "Estimation of the Park–Ang damage index for planar multi-storey frames using equivalent single-degree systems", *Engineering Structures*, 33(9), 2509-2524.

Habibi, A. R., Izadpanah, M. and Yazdani, A. (2013). "Inelastic damage analysis of RCMRFS using pushover method", *Iranian Journal of Science and Technology, Transactions of Civil Engineering*, 37(C2), 345-352.

Hancock, J. and Bommer, J.J. (2006). "A state-of-knowledge review of the influence of strong-motion duration on structural damage", *Earthquake Spectra*, 22(3), 827-845.

IBC-2012, (2012). *International building code*, International Code Council, Country Club Hills: USA.

Jeong, S. and Elnashai, A.S. (2007). "Fragility relationships for torsionally-imbalanced buildings using three-dimensional damage characterization", *Engineering Structures*, 29, 2172-2182.

Kamaris, G.S., Hatzigeorgiou, G.D. and Beskos, D.E. (2013). "A new damage index for plane steel frames exhibiting strength and stiffness degradation under seismic motion", *Engineering Structures*, 46, 727-736.

Khatib, I.F., Mahin, S.A. and Pister, K.S. (1988). "Seismic behavior of concentrically braced steel frames", Report No. UCB/EERC-88/01, Earthquake Engineering Research Center, University of California, Berkeley.

Kunnath, S.K., Reinhorn, A.M. and Lobo, R.F. (1992). "IDARC Version 3: A program for the inelastic damage analysis of RC structures", Technical Report NCEER-92-0022, National Center for Earthquake Engineering Research, State University of New York, Buffalo, NY.

Mazzoni, S., McKenna, F., Scott, M.H. and Fenves, GL. (2016). *OpenSEES command language manual*, Pacific Earthquake Engineering Research Center, http://opensees.berkeley.edu.

Nazri, F.M. and Alexander, N.A. (2012). "Determining yield and ultimate loads for MRF buildings", *Proceedings of the ICE - Structures and Buildings*.

Nazri, FM. and Alexander, NA. (2014). "Exploring the relationship between earthquake intensity and building damage using single and multi-degree of freedom models", *Canadian Journal of Civil Engineering*, 41(4), 343-356.

Park, Y.J. and Ang, A.H.S. (1985). "Mechanistic seismic damage model for RC", *Journal of Structural Engineering*, 111(4), 722-739.

Park, Y.J., Ang, A.H. and Wen, Y.K. (1987). "Damage-limiting aseismic design of buildings", *Earthquake Spectra*, 3(1), 1-26.

Poljansek, K. and Fajfar, P. (2008). "A new damage model for the seismic damage assessment of reinforced concrete frame structures", *14th World Conference on Earthquake Engineering*, Beijing, China.

Powell, H.G .and Allahabadi, R. (1988). "Seismic damage prediction by deterministic methods: Concepts and Procedures", *Earthquake Engineering and Structural Dynamics*, 16, 719-734.

Rajeev, P. and Wijesundara, K.K. (2014). "Energy-based damage index for concentrically braced steel structure using continuous wavelet transform", *Journal of Constructional Steel Research*, 103, 241-250.

Reinhorn, A.M. and Valles, R.E. (1995). "Damage evaluation in inelastic response of structures: a deterministic approach", Report No. NCEER-95-xxxx, National Centre for Earthquake Engineering Research, State University of New York at Buffalo.

Rodriguez, M.E. and Padilla, D. (2009). "A damage index for the seismic analysis of reinforced concrete members", *Journal of Earthquake Engineering*, 13(3), 364-383.

Rodriguez, M.E. (2015). "Evaluation of a proposed damage index for a set of earthquakes", *Earthquake Engineering and Structural Dynamics*, 44(8), 1255-1270.

Sabelli, R. (2001). "Research on improving the design and analysis of earthquake resistant steel-braced frames", NEHRP Fellowship Report No. PF2000-9, Earthquake Engineering Research Institute, Oakland, California.

SeismoMatch, (2016). "A computer program for adjusting earthquake records to match a specific target response spectrum", Available from: http://www.seismosoft.com.

Shahraki, H. and Shabakhty, N. (2015). "Seismic performance reliability of RC structures: Application of response surface method and systemic approach", *Civil Engineering Infrastructures Journal*, 48(1), 47-68.

Tremblay, R. and Tirca, L. (2003). "Behavior and design of multi-story zipper concentrically braced steel frames for the mitigation of soft-story response", *Proceedings of the Conference on Behavior of Steel Structures in Seismic Areas*.

Uriz, P. and Mahin, S. (2004). "Summary of test results for UC Berkeley special concentric braced frame specimen No. 1 (SCBF)", http://www.ce.berkeley.edu/_patxi/SCBF/publications/PrelimSCBFtestResul.pdf.

Usami, T. and Kumar, S. (1998). "Inelastic seismic design verification method for steel bridge piers using a damage index based hysteretic model", *Engineering Structures*, 20(4), 472-480.

Vaseghi, J., Esmaeilnia, M. and Ganjavi, B. (2015). "Achievement of minimum seismic damage for zipper braced frames based on uniform deformations theory", *Journal of Rehabilitation in Civil Engineering*, 3(1), 43-60.

Vaseghi, J., Esmaeilnia, M. and Ganjavi, B. (2016). "Ductility reduction factor for zipper-braced frames", *European Journal of Environmental and Civil Engineering*, DOI: 10.1080/19648189.2016.1262283.

Williams, S.M. and Sexsmith, G.R. (1995). "Seismic damage indices for concrete structures: A state of the art review", *Earthquake Spectra*, 11(2), 319-349.

Yang, C., Leon, R. and DesRoches, R. (2008). "Design and behavior of zipper-braced frames", *Engineering Structures*, 30, 1092-1100.