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Original Paper

Prioritization and Assessment of the Existing Damage Indices in Steel Moment-Resisting Framed Structures

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Abstract

Seismic behavior assessment of framed structures is necessary for life and financial damage estimation. During recent years, as an earthquake engineering progresses, knowledge and experience related to seismic responses have been developed. The new seismic behavior assessment methods have been proposed so it is indispensable to define a formulation as a damage index, which damage amount has been quantified and qualified during earthquakes. In most cases, damage indices are dimensionless parameters intended to range between zero for the undamaged state and unity for the collapsed state of a member or an entire structure and other values show different damage states. In this paper, 4 new steel moment-resisting frames with different height and regular plant were supposed and designed according to ASCE7-2010. The necessary parameters were obtained for damage indices such as local indices: interstory drift, maximum roof displacement, banon failure, kinematic, banon normalized cumulative rotation, cumulative plastic rotation and ductility and also global indices: roufaiel and meyer, papadopoulos, sozen, rosenblueth, ductility, hysteretic energy, Park-Ang and base shear, and finally story indices: banon failure (weighted index) and interstory rotation hence they have been derived and calculated under the effect of far-fault ground motion records by non-linear dynamic time history analysis using SeismoStruct V.7.0.6. Finally, prioritization of the existing damage indices, is defined based on more conservative values in terms of more damageability rate in the studied models. Then as a result banon failure, interstory drift and park-ang are more conservative local, storey and global indicators respectively among the existing damage indices.

Keywords: Steel Moment-Resisting Frame, Non-linear Dynamic Time History Analysis, Damage Index, Prioritization, Far-Fault Ground Motion Records, SeismoStruct V. 7.0.6.

1. Introduction

The concepts of damage and damageability rate play a central role in the seismic design of structures. It is approved that standard design approaches, based on the concept of the force reduction factor, even if adequate in most practical cases, do not result in structures possessing uniform and rationally defined safety and performance levels. For this reason, the concept of damage indices or damage indicators has become popular. Damage indices proposed in published technical literature quantify local and global structural damage of buildings, subject to base excitations, on a scale ranging from zero to unity; where zero score represents undamaged state and unity represents collapse damage state of buildings.

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This quantification helps in assessing seismic performance of building through analytical methods and helps in several applications such as selecting retrofitting options. However, damage indices are not adequately correlated to post-earthquake damage states that are defined based on observational methods.

In the last decades many methodologies on seismic damage prediction are developed. Hence, a great effort has been made to improve the current earthquake resistant design methods in order not only to avoid collapse under destructive earthquakes but also to limit the damage under moderate earthquakes. Furthermore, the new design philosophy is tending to multi-level probabilistic structural performance criteria, replacing completely the simple force strength approach. However, the implementation of all these new concepts requires the definition of qualitative damage index.

The vulnerability of many existing structures may be due to structural weaknesses and low ductility. Common weaknesses in the structural system are due to incomplete load path; strength and stiffness discontinuities, plan and height irregularities; weak column/strong beam, and other eccentricities. Low ductility detailing is characterized as insufficient shear reinforcement, inadequate confinement and insufficient anchorages and other detailing. The state of damage of a component, a story, or the whole structure may be represented by an index. The damage index is used as an indicator to describe the state of the lateral load-carrying capacity and the reserve capacity of existing structures. Thus, the study on damage index and its availability is necessary. Some damage indices are calculated for each component of the building as local damage indices. The component damage indices may be integrated using a weighting procedure to provide the global damage index for the structure. These damage indices have been formulated using response parameters of the structure that are obtained through analytical evaluation of structural response. The typical response-based damage indices include ductility ratio, inter-story drift, slope ratio, maximum drift, flexural damage ratio, low cycle fatigue, final softening index and Park-Ang index. The damage indices such as inter-story drift and maximum drift are fundamental and essential for representing the displacement or deformation.

In this paper, the methodology is examined by performing numerous non-linear dynamic time history analyses on a set of 2D steel moment-resisting frame archetypical structures (2, 5, 8 and 12-story) representing low-rise to high-rise buildings. To do this task, normalized responses are defined as a damage index subjected to 4 far-fault ground motion records. The outcomes (presented in tables) provide valuable answers to the targeted topic discussed above. Then it was tried to calculate, compare and prioritize the various existing Local/Global damage indices derived technical literature in steel moment-resisting frames with intermediate ductility and regular plant under the effect of far-fault ground motion records by non-linear dynamic time history analysis using SeismoStruct V7.0.6 software [1].

2. Damage Assessment

In the last decades many methodologies on seismic hazard analysis and damage prediction are developed. In seismic design, having life safety is really important while earthquake happens and controlling the damages of the structure which is repairable. In performance based design codes, the levels of performance are defined based on the inelastic deformation of the structural elements, so it is necessary to have an appropriate relation between the levels of performance and the damages of the elements. For the damage assessment in the elements, criterions are defined as damage indices that they are based on FEMA-356 code and HAZUS report and local /global damage rate are specified according to performance levels. Damage assessments are introduced in the following sections according to FEMA-356 code and HAZUS report.

2.1. Damage Assessment Based on FEMA-356

FEMA356 [2] defines the structural performance levels:

1. Linear Limit: The structure response restricted to linear limit.

2. Immediate Occupancy Level: The structure will be safe to occupy after the earthquake.

3. Damage Control Level Range: A damage state between life safety and immediate occupancy performance level.

4. Life safety Level: Structure is damaged but retains a margin against onset of partial or total collapse.

5. Limited Safety Range: A damage state between collapse prevention and life safety performance level.

6. Collapse Prevention Level: The structure continues to support gravity loads but retains no collapse.

7. Collapsed level: The structural performance level is assessed by two damage variables:

A. Interstory Drift Ratio: Interstory Drift ratio is the top deflection of a structure over the height of the structure and is a natural global index.

B. Plastic Deformation Normalized by Yield Deformation: The maximum local indices of the structure should not be greater than the limits defined by the standard. To compare the FEMA discrete performance levels with damage indices, each FEMA-356 performance level is tentatively assigned to a value between zero and unity.

2.2. Damage Assessment Based on HAZUS

The report (HAZUS) (1999) [3] estimates the extent of damages related to structural members (relative displacement) and non-structural members (relative displacement and acceleration). FEMA[1] was developed in order to estimate earthquake. Its main goal is to provide an instruction to estimate the risks of earthquakes on a regional scale. According to HAZUS report, the structures are divided into 36 with regard to construction materials groups. The groups are divided into three subgroups in terms of the number of story: buildings with (1-3 story) is (low-rise), buildings with (4-8 story) is (mid-rise) and over 8 story is (high-rise). The overall classification of structures from the point of view of the HAZUS report includes residential, commercial, industrial, agricultural, religious, public and educational structures and table 1 shows critical interstory drift ratio and related damage states of framed structures exactly according HAZUS report.

Table 1. Interstory Drift Ratio of steel moment-resisting frame based on HAZUS [3]

Damage State	Low-Rise	Mid-Rise	High-Rise
Slight	0.006	0.004	0.003
Moderate	0.010	0.0066	0.005
Extensive	0.024	0.016	0.012
Complete	0.06	0.04	0.03

[1] The Federal Emergency Management Association

3. Damage Index Definition

In today's world, there is much talk about seismic vulnerability, its features and the necessity of study. The main problem that is faced in today's world is knowledge about proper quantification of damage. The response is obtained after the ground motion has been imparted on the structure. Despite of making significant progress in the field of seismic design codes for dynamic analysis, there is a lack of progress in the quantification of damage. The damage criterion should include large displacements as well as the effect of repeated cyclic loading. An energy-based damage model is used to assess the damage index for the structure taking into account the effect of repeated cycles.

Damage index is introduced for seismic damage assessment of the structure in order to quantify degree of damage numerically. The concept of damage index can provide the means to quantify damage and relate it to costs and other consequences such as potential risk after earthquake. Hence, damage index can play an important role in retrofit and rehabilitate decision-making and disaster planning in earthquake region. Two kinds of damage index are introduced [4].

Several physical responses of structures have been used as indicators of damage at the structural level, which are called damage parameters. Each damage index uses specific damage parameters to categorize damage states.

Damage parameters could be classified as following parameters in structural responses:

1. Plastic deformation of elements or structure.
2. Energy Dissipation through hysteretic behavior in the elements.
3. Low cyclic fatigue of the elements.
4. Dynamical parameter variation of structure such as the first natural period of structure.

Damage indices are usually normalized so that their value is equal to zero when there is no damage and is equal to unity when total collapse or complete failure occurs. A damage index can involve a combination of one or more damage variables in its calculation. As a result, in order to calculate damage indices, damage parameters should also be normalized. The normalization of damage variables could be based on one of the following approaches:

A. the demand versus capacity approach is based on estimation of certain demand on a structure, sub-structure or member, and estimation of corresponding capacity. This kind of normalization was more popular. Several well-known indices like Park and Ang [13] uses this kind of normalization.

B. the calculated degradation of a certain structural parameter, like stiffness or energy dissipation or natural period of structure, is compared with a critical value, and it is usually expressed as a percentage of initial value corresponding to the undamaged state or the last stage value as a damaged state.

Damage indices based on the strength are simple and do not require an analysis of the structural responses and depend on the geometric properties of elements such as cross-section of the beams, columns, braces and steel and reinforced concrete shear walls and their materials. These types of damage indices should be calibrated by the observed damage using large real database or the results of numerical analysis of structures. The damage index based on strength was first proposed by Shiga et al. (1968) [5] and later by Yang (1980) [6]. In the damage evaluation method, based on structural response, a relatively complex analysis is required, but less information is needed to calibrate the results. This method requires detailed information on constructive models, materials and descriptions of site-compatible ground motion records [7]. Fig.1 shows damage index classification.

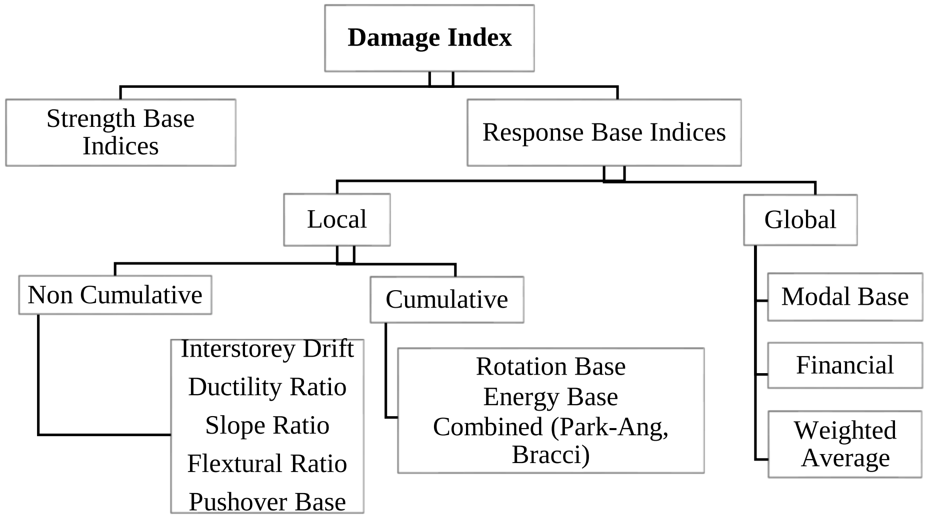


Figure 1. Damage Index Classification [8]

3.1 Damage Index Background

The background of activities in the field of structural damage index dated back to the first 70s. Shiga et al. (1968) introduced damage index according to strength for the first time [1]. Whitman (1972) stated the earthquake-induced damage index with the cost of repair to the cost of rebuilding in different degrees of ground motion with a modified scale [4]. Okada et al. (1974) presented a new method for assessing the seismic safety of reinforced concrete structures [9]. Yao (1975) also introduced the damage index based on relative displacement [10]. Bertero and Bresler (1977) described the definitions of local and global damage of structure [11]. Banon et al. (1981) presented the damage index based on the initial stiffness ratio at maximum displacement of the stories, and in 1982, the damage model was defined based on the deformation factors [12]. Krawinkler et al. (1983) proposed an index for cumulative damage estimation, which is directly related to structural performance parameter, degree of plastic deformation and total number of cyclic movements [13]. Park et al. (1984) developed a major evaluation method of seismic vulnerability [14]. Park and Ang (1985) presented a new method based on maximum deformation of the member and its integration with the maximum absorbed energy at yield point [15]. Roufaiel and Meyer (1987), evaluated the seismicity of steel and RC structures, and they defined an index for the entire structure according to structural characteristics [16]. Powell and Allah Abadi (1988) presented a method for calculating the damage index based on comparison of structural capacities during earthquakes [17]. This subject was expressed by Cosenza (1993) with the same formulation but it was based on the ductility and absorbed hysteresis energy in the structure [18]. Bracci et al. (1989) formulated the global damage index for steel and RC structures [19].

Krawinkler and Nasser (1992) considered the damage of structural elements based on ductility and cumulative damage indices in steel structures [20]. Oghlo et al. (1994) presented an index based on frequency variation of the first vibrational mode due to the reduction of stiffness and strength. By studying the hysteresis behavior, they predicted local and global damage of first mode [21]. Daali and Korol (1996) proposed a damage index based on the Park index [22]. Ghobarah and Abu al-Fattah (1997) proposed a special damage index, based on structural responses and stiffness parameter was obtained by nonlinear static analysis (pushover) before, during and after earthquakes [23]. Ghobarah and El-Attar (1998) presented a new method for assessing the concentration of damage in RC frames by combined damage index [24].

In 1999, in Australia, a researcher group presented a linear model for calculating the damage index based on building cost and introduced empirical coefficients in analysis. In the same year, after the earthquake of Taiwan, John Miyakoshi compared Chi-Chi earthquake (1995) damage with Kobe earthquake (1995), as a result a new relationship was obtained to calculate the damage of school buildings by damage index based on an empirical test [25]. With the help of Park and Ang (1985), Mikami and Imura (2000) presented a new relationship in which the maximum flexure and strength of steel were considered in the elastic range [26]. Papadopoulos et al. (2004), presented a simple combined damage index to evaluate vulnerability of structures with regard to soil-structure interaction [27]. Abbas Nia and Barghi (2004), examined 25 reinforced concrete columns with a specific loading history by Park - Ang damage index [28]. Kianfar, Estekanchi and Vafaei (2004) examined the performance levels of steel frames with 3 and 7 stories by various damage indices [29]. Gerami and Daneshjoo (2006) examined story drift as an indicator of cumulative damage in steel moment frames [30]. Jeong and Elnashai (2006) developed fragility curves for irregular structures in a plan and then introduced 3 dimensional damage index [31]. Vaseghi Amiri et al. (2008) studied three RC frames with 8, 12 and 15 stories with RC shear walls, and then evaluated the distribution of damage index during strong earthquakes [32]. Barghi and Rajabi (2010) developed Park-Ang Damage Model by testing on reinforced concrete columns with flexural fractures with regard to cyclic loading and using laboratory results of 95 columns. Sadeghi (2011) proposed a simple and precise damage index for evaluating structural damage in a cyclic loading model [33]. Roanagh (2014) presented the relationship between seismic parameters of movement in far- faults ground motions and then developed damage index for short RC frames [34]. Kazemi et al. (2013) corrected the Elnashai Damage Index by modifying it and used this index to evaluate the damage of irregular structures with steel frame and RC shear walls in Holy Mashhad city [8]. Zhang et al. (2014) performed sensitivity analysis of the correlation function based on damage index and its application in detecting damage [35]. Morik et al. (2014) proposed a combined damage index for assessing the failure of regular structures in plan [36]. In addition, Rajeev et al. (2014) proposed a damage index based on absorbed energy in Concentric Braced Frames (CBF) [1]. Shabani and Abdollahzadeh (2015), evaluated vulnerability of steel frames with viscoelastic dampers by energy damage index under different earthquake records [37]. Emami et al. (2015) assessed Park-Ang damage index (local, story and global) for four RC moment frames with 4, 8 and 12 stories under 14 near-fault ground motion records [38]. Abbasi and Mirzai (2016) studied the seismic probability sensitivity of RC frames with 7 and 10 stories by fragility curves [39]. Vosoughi et al. (2016) evaluated and compared the lightweight steel framed (LSF) structures considering soil-structure interaction, quantitatively and qualitatively, using the Papadopoulos damage index [40]. Saleemudin et al. (2017) evaluated seismic damage indices of RC framed structures using non-linear static analyses [41]. Liu et al. (2017) assessed correlation between global damage index and local damage indexes for the seismic performance of framework and the results show that the global damage index is generally closed to weighted value of local damage indexes which taking the ratio of hysteretic energy dissipated by plastic hinge to total value dissipated by frame as weight. Therefore, the weighted value of local damage indexes could be used to estimate the global damage index of structure [42].

4. Methodology

Seismic vulnerability assessment and calculating the damage indices requires sufficient real data or laboratory tests that need to be validated but since this information is usually inadequate and costly, so that using numerical and analytical modelling on selected structures is discussed. In this paper, it is generally allowed to study a 2-dimensional (2D) model and then generalizing the results to a 3-dimensional (3D) model on condition that no remarkable torsion happens in the 3D model (Iranian Loading Code [29]), 2D frames have been used in this research. so to compare and prioritize the damage indices for evaluating the damages of members, stories and the entire buildings (Local, Story and Global Indices) hence, the 2D intermediate ductility residential steel moment-resisting frames' Designed [43] with 3 bays in 4 types of 2, 5, 8 and 12 storeys that they were chosen according to the report "HAZUS"[3]. The buildings were designed based on the American code (ASCE7-2010) [44] and for design sections; the LRFD method based on AISC 360[45] was done. Then design control was done not only stress but also drift and the gravity loads such as dead and live loads and load combinations (DL+0.2LL) were determined according to ASCE7-2010. These frames sustain the mean dead load of 300 kg/m2 in each floor area and 250 kg/m2 for roof area. The mean intensity of live load for typical floors and roof is assumed as 200 kg/m2 and 150 kg/m2, respectively. The difference in loads between typical floors and roof arises from different construction details for floors and roof. The buildings have regular rectangular plan as shown in Figure.1. Then a lateral resistant system is a moment frame in the X and Y direction therefore HEB sections for beams and BOX sections for columns were used in the modelling, analysis and design.

The specifications of the studied frames and the regular plan and lengths of the bays were selected based on Kumar et al. [46], shown in Fig.2. The elevation of selected intermediate frames according to height was shown in Fig.3. Moreover, the configuration of the frames and designed sections were implied in table 2 and Fig.4

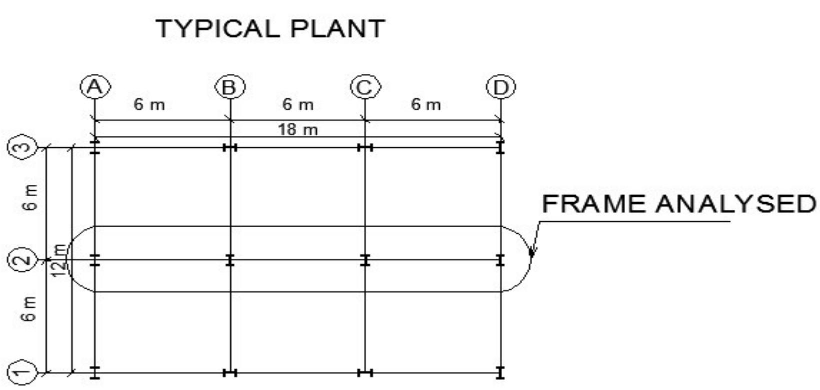
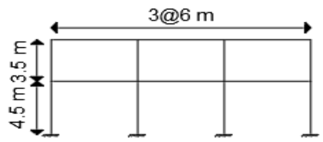
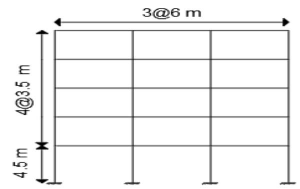


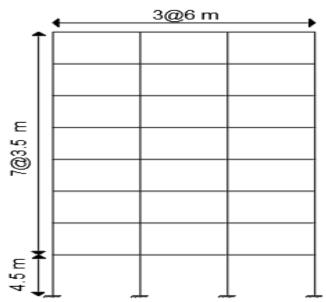
Figure 2. Typical plant of steel moment resisting frames [46]



(A)



(B)



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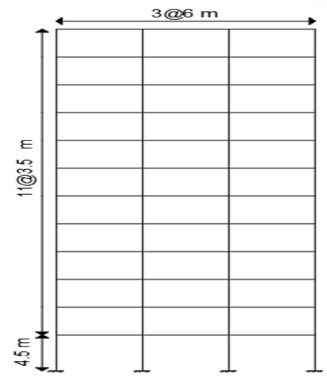
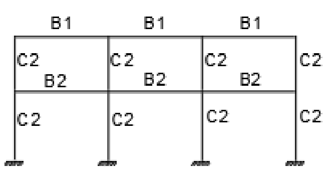
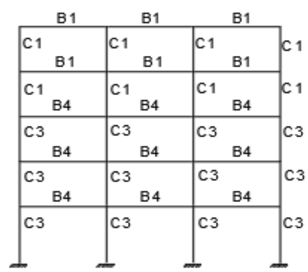


(D)

Figure 3. Geometrical specification of steel moment resisting frames (A, B, C and D represented frames with variable height)



(A)



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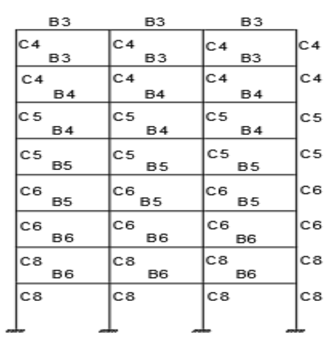
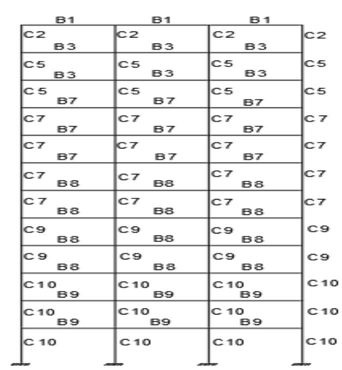


(C)



(D)

Figure 4. Design sections of steel moment resisting frames (A, B, C and D represented frames with variable height)

Table 2. Introduction of design sections of steel moment resisting frames' elements

No. Beam	Design Section	No. Column	Design Section
B1	HE220B	C1	BOX200X200X15
B2	HE240B	C2	BOX200X200X20
B3	HE260B	C3	BOX200X200X25
B4	HE280B	C4	BOX220X220X10
B5	HE300B	C5	BOX220X220X20
B6	HE320B	C6	BOX240X240X40
B7	HE340B	C7	BOX250X250X20
B8	HE360B	C8	BOX280X280X35
B9	HE400B	C9	BOX300X300X20
		C10	BOX300X300X35

The main period of these steel frames extracted by softwares Etabs V.2013 and SeismoStruct V.7.0.6 to verify modelling and then values were shown in table 3. The maximum percent of differential ratio is less than 12%. In addition, the type of construction materials is Mild Steel that is introduced as ST-37, its weight per volume is 7850 kg/m^3 with a yield stress 240 MPa and an ultimate stress 360 MPa and then Elasticity Modulus, Poisson's Ratio and Shear Modulus are 200GPa, 0.3 and 79.3 GPa respectively.

Table 3. Modal specifications of the steel frames extracted by Etabs V. 2013 and SeismoStruct V.7.0.6

No. Story	Height(m)	Etabs Period(Sec)	SeismoStructPeriod(Sec)	Differential Ratio
2	8	0.58	0.65	0.11
5	18.5	0.98	1.09	0.1
8	29	1.60	1.50	0.07
12	43	1.78	1.75	0.02

4.1 Non-linear Dynamic Time History Analysis

In this paper, Finite Element software SeismoStruct V.6.0.7 [50] is used for non-linear dynamic time history analysis. This software is provided by Pinto and it has a simple and interesting graphical environment. The non-linear behavior of "Fiber Element" has been used to define the plasticity distribution in the entire length of the member such as beams and columns. The number of fiber sections is 200, and the number of Gaussian integrations along the beam and column elements is 5.

The non-linear cyclic behavior of materials is used in these frames is the Menegotto-Pinto (1973) behavioural model and it is utilized in the infrFB type as structural beams and columns in according to Fig. 5 [48]. The effect of large deformations in the applied model and the various structural responses have been measured and recorded at various time steps.

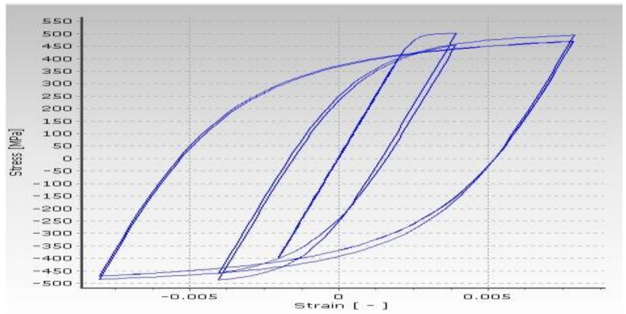


Figure 5. Cyclic behaviour model of steel [50]

4.2 Finite Element Software "SeismoStruct"

SeismoStruct is an award-winning Finite Element package capable of predicting the large displacement behaviour of space frames (2D and 3D) under static or dynamic loading, taking into account both geometric nonlinearities and material inelasticity. Concrete, steel, FRP[2] and SMA[3] material models are available, together with a large library of 3D elements that may be used with a wide variety of pre-defined steel, concrete and composite section configurations. The program has been extensively quality-checked and validated, as described in its Verification Report [47]. In this paper, SeismoStruct V7.0.6 is used in order to model and evaluate the existing damage indices of structural prototypes.

4.3 Ground Motion Records Selection

One of the important and effective parameters in the structural responses is the input energy of the accelograms in non-linear dynamic analysis. As a result, the conclusions of the vulnerability assessment will depend on the acceleration of the input records. Totally After Loma Prieta 1989 earthquake, the ground motions have been divided in 3 groups: 1- Near-field earthquakes: the distance between site and fault is less than 20 Km. 2- Mid-field earthquakes: the distance between site and fault is between 20 Km to 50 Km. 3- Far-field earthquakes: the distance between site and fault is larger than 50 Km [5]. so In this paper, the far-fault ground motion records were obtained then SeismoSignal and Excel softwares are used to process earthquake data as well as the results of analyses. SeismoSignal software is one of SeismoSoft's software set [47]. This software allows processing of earthquake records and Excel software is also used to scale earthquakes, final processing of non-linear analysis outputs and plotting the diagrams. Fig 7, 8 and 9 shows frequency content, scaling accelograms and acceleration response spectrums. In this paper, according to Tables 4, 5 and 6. Four far-fault ground motion records with magnitudes of 6.5 to 7.5 were applied on soil type II with shear velocity wave of $375 < V_s < 750$ from earthquake data bank (PEER) [4]Extracted [49]. In addition, the distance between the recording stations of these accelograms is selected from the earthquake surface centre of 30 to 100 kilometres. Accelograms are scaled based on a maximum acceleration of 30% gravity acceleration (design acceleration), so that a better comparison of present models can be made.

[2] Fiber Reinforced Polymer
[3] Shape Memory Alloy
[4] The Pacific Earthquake Engineering Research Centre

Table 4. Ground motion records selection criteria

Accelerogram Location	Ground Level	Source Distance (Km)	30-100
Motion Duration (Sec)	≥10	Magnitude (Ms)	6/5-7/5

Table 5. Earthquake events

ID		Earthquake	Country	Component	Magnitude (Ms)	Fault Type	Station	Distance (Km)	Date
Far-Field	N1	Tabas	IRAN	FER-L1	7.4	Reverse	Ferdows	91.14	16.09.1978
	N2	Manjil	IRAN	185066	7.4	Strike-slip	Qazvin	49.97	20.06.1990
	N3	Sanfernando	USA	PPP-270	6.6	Reverse	Pearblossom Pump	37.4	09.02.1971
	N4	Kobe	JAPAN	HIK-000	6.9	Strike-slip	HIK	95.72	16.01.1995

Table 6. Seismic characteristics of ground motion records

ID		Earthquake	PGA(g)	PGV(cm/sec)	PGD(cm)	PGV/PGA(sec)
FarField	N1	Tabas	0.105	7.144	4.676	0.069
	N2	Manjil	0.184	15.284	4.196	0.085
	N3	Sanfernando	0.138	5.499	1.932	0.041
	N4	Kobe	0.139	14.720	2.298	0.107

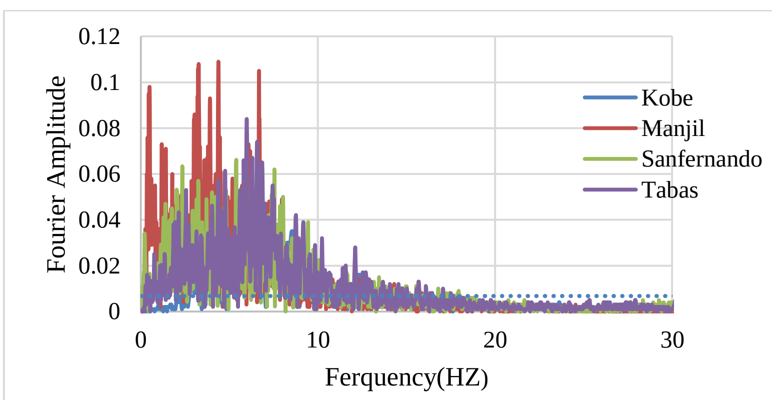


Figure 6. Frequency content of the studied ground motion records

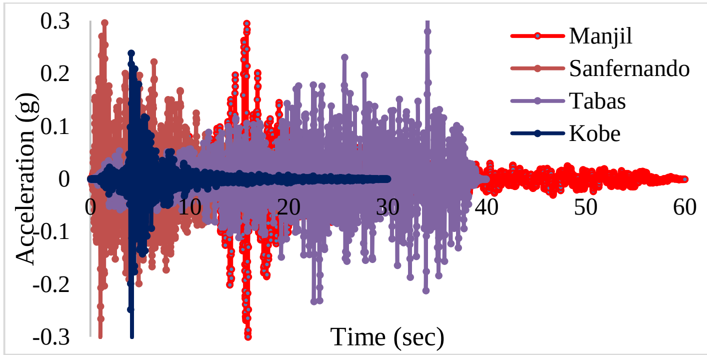


Figure 7. Studied accelograms scaled to base design acceleration

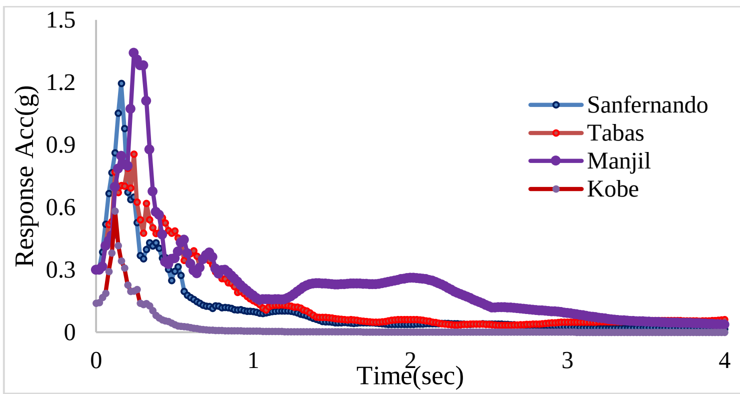


Figure 8. Elastic acceleration response spectrum scaled to base design acceleration with 5% damping

5. Studied Damage Indices

First, a brief overview of the theory of damage index such as studied local and global damage indices is presented and then the necessary results of this study are presented in the following sections.

5.1 Local Damage Indices

Local damage indices are indicators of damage for a part of a structure, such as an element or even a story, while a global index gives an estimate of overall damage to the structure. In order to determine an index for the entire structure from local indices, we should weigh these local values into a global parameter. Local indices that indicate damage in a member or a connection or a joint under an earthquake loading [42].

5.1.1 Interstory Drift Index

Interstory drift ratio, which is the most practical index between structural engineers and designers, it could the necessary index for evaluating the status of damaged structures. This index is defined as the ratio between the maximum displacement of the structure at the target point and the story height, and it is a naturally global index. Drift damage index of the story is in accordance with an equation.1. and it can be calculated using static or dynamic analysis (Pushover and Time History):

$$DI_{drift} = \frac{\Delta_m}{h} \tag{1}$$

Where Δ_m: the relative displacement of the story (m), h is the height of the story, and DI_{drift}: the relative displacement ratio of the story (m) that equals to drift [2].

5.1.2 Maximum Roof Displacement Index

The collapse state of structural and non-structural elements in severe earthquakes are due to excessive displacement created on the building stories.

The reaction of buildings in heavy earthquakes is always overcoming the limit of yielding and tolerance of large strains, and in accordance with the base shear-displacement diagram of the structure, in this area, changes in the resistance are negligible and controlling the behavior of the building, deformation or displacement. During an earthquake, if the roof or one of the stories moves beyond a certain limit, the structure is collapsed because in many cases, large displacements are equivalent to heavy and massive damage in the stories. Therefore, attention to maximum roof displacement is important as a suitable measure for designing structures, especially in tall and special buildings [2].

5.1.3 Banon Failure index

In many structural members, non-linear deformations are formed in the form of connections and joints at the ends of the members. Therefore, for the vulnerability evaluation of such members, we can use other parameters such as displacement and curvature instead of rotation. This damage index is defined by equation.2.

$$DI = \frac{\phi_m}{\phi_u} \tag{2}$$

In the above equation, ϕ_m andϕ_u, respectively, represent the maximum rotation and the final rotation of members [12].

5.1.4 Kinematic Index

Other damage indices, which are used in the studies and in particular to compare the performance of the damage indices more reliable, is a kinematic damage index that defined according to an equation.3.

$$DI = \frac{\phi_m-\phi_y}{\phi_u-\phi_y} \tag{3}$$

ϕ_m, represents the maximum plastic deformation of the model during an earthquake; ϕ_u, the ultimate plastic deformation; ϕ_y, the yield plastic deformation. The above equation is also expressed in terms of the angles of the rotation of the elements [30].

5.1.5 Banon Normalized Cumulative Rotation Index

Banon and Veneziano (1982), according to an equation.4, introduced this damage index. The parameter ϕ_{im} represents the maximum rotation in the i cycle.

$$DI = \frac{\Sigma(\phi_{im}-\phi_y)}{\phi_u} \tag{4}$$

This measurement was performed for a number of cyclic load tests, in which they were essentially bending, and for a number of important axial loads. If there were large correlations, the index values in the collapse indicated significant dispersion [12].

5.1.6 Cumulative Plastic Rotation Index

The Cumulative Plastic Rotation Damage Index was introduced by Banon (1981). The parameter θ_{pi} represents the plastic chord rotation in the cycle (i). This damage index is presented below by an equation.5. [30].

$$\theta_{pa} = \sum_{i=1}^{N} \theta_{pi} \tag{5}$$

5.1.7 Ductility Index

Newmark and Rosenblueth (1971) introduced the coefficient of ductility, and the parameter ϕ_m represents the maximum rotation and the parameterϕ_y represents the yielding rotation in the equation.6.

$$\mu_r(\phi) = \frac{\phi_m}{\phi_y} \tag{6}$$

It is necessary to pay attention that this damage index is completely applicable for structural elements [50].

5.1.8 Interstory Rotation Index

Interstory rotation index has formulation as follows. It should be noted that this index can be computed by using dynamic and static analysis and it is calculated as an equation.7.

$$\phi *_{\mathrm{i}} = \frac{\phi \mathrm{i}}{\mathrm{h}} \tag{7}$$

Where ϕ_i: the story rotation, h is the height of the story and $\phi *_{\mathrm{i}}$: the relative rotation of the i story.

5.1.9 Banon Failure (Weighted Index)

This damage index is based on the weighting Indices of the local damage to the global, in which $D_{story,i}^{b+1}$represents the local damage to the i story and $W_{story,i}$ represents the weight of the structural element, and the global damage index is following as an equation.8.

$$D_{global} = \frac{\sum_{story,i=1}^{N} W_{story,i} D_{story,i}^{b+1}}{\sum_{story,i=1}^{N} W_{story,i} D_{story,i}^{b}} \tag{8}$$

In the easier case, the above equation was alternated to an equation.9.

$$DI = \sum_{i=1}^{N} \frac{{d_i}^2}{d_i} \tag{9}$$

Bracci et al., Defined the weighted gravity loads endured by element i, divided by the weight of the structure. According to this definition, weighing is much more affected on the base parts of a structure than the higher stories, since the possibility of complete collapse of the structure due to this damage will be much greater [19].

5.2 Global Damage Indices

Global damage indices take into account the whole structure and its characteristics and provide information about the global damage state as a function of the distribution and severity of local damages.

If a structure is statically determined, local failure in most damaged members is sufficient to determine the structural failure condition. However, for uncertain structures, it is necessary to define global indices in order to consider the extent and distribution of local damages. The indicators can be determined in two ways. Firstly, the average weights of local indices are considered for all structural members. In the second method, parameters are used that reflect the behavior of the structure (such as periodic and median frequencies). The indices can be obtained by subtracting local indices of the structure, or by comparing the modal properties of the structure before and after or during the earthquake. Damage analysis is done according to global indices of localization; local indices are derived and have the same problems as local damage indices [44].

5.2.1 Roufaiel and Meyer Index

Roufaiel and Meyer (1987) proposed the following damage index for steel and reinforced concrete framed structures. The Roufaiel and Meyer damage index is followed as an equation.10.

$$DI = \frac{d_r - d_y}{d_f - d_y} \tag{10}$$

d_r: Maximum displacement in earthquake; d_f: final displacement;d_y: yield displacement [16].

5.2.2 Papadopoulos Index

This relationship was presented by Papadopoulos and colleagues in a numerical-laboratory relationship. Hence, the use of this relationship challenges designers and researchers because this index is simple. In this index, seismic geotechnical effects are based on the action and it is given as an equation.11.

$$GDP_r = r.\frac{d_r - d_y}{d_f - d_y} \tag{11}$$

In the equation.11. r is the correction coefficient, which indicates the seismic geotechnical effects that since the project was done on soil type II, so the value of correction coefficient is 2 [27].

5.2.3 Rosenblueth Index

Newmark and Rosenblueth presented the equation.12 to assess the structural demand:

$$DI = \frac{\delta_a}{\delta_u} \tag{12}$$

In this case, δ_a and $\delta_u$are the maximum and an ultimate displacement of structure respectively [50].

5.2.4 Ductility Index

Newmark and Rosenblueth presented the equation.12 to assess the structural capacity:

$$DI = \frac{\delta_a}{\delta_y} \tag{13}$$

Where δ_a and δ_y are the maximum and an ultimate displacement of structure respectively. In this case, the maximum displacement of the story is calculated in the order of the structural damage of the building. It is usually assumed that failure occurs when the ductility of the requirement is exceeded by the permeability of the structure [50].

5.2.5 Sozen Index

Regarding the quantification of this criterion, Sozen (1981) suggests that the percentage of damage and relative displacement is suggested as the following: MIDR is the maximum relative displacement of the story. This index is defined as an equation.14.

Damage (%) =50*(MIDR%)-25 (14)

From the analysis of experimental data on small-scale components and structures, it is indicated that the relative displacement of stories smaller than 1 percent, larger than 4 percent and larger than 6 percent, respectively, cause the destruction of undamaged members, damaged members that need to be repaired and the building collapse [51].

5.2.6 Hysteric Energy Index

The value of hysteretic energy and strain energy are known from the response history for energy for the building frames in the study for a set of earthquakes. Tables to contain the hysteretic energy normalized with the strain energy for the building frameworks. Hysteretic energy is related with the cumulative ductility and can be estimated through the simple relation. Further, the same expression has been used for finding total energy dissipated (Hi) during reversal of stresses due to varying earthquake ground motions and (Eih) shows hysteric energy during seismic loading [52].

$$DI = \frac{\sum_1^n E_{ih}}{H_i} \tag{15}$$

5.2.7 Park-Ang Index

One of the most important damage models is the Park & Ang damage index [15]. It shows the damage of reinforced concrete elements as a combination of maximum deformations and the absorbed cyclic energy as:

$$DI_{P\,\&\,A} = \left(\frac{\delta m}{\delta u}\right) + \left(\frac{\beta}{\delta u * P}\right)\int dE \tag{16}$$

Where δm and δu are the maximum and ultimate deformations respectively, yield strength of element denotes as P and dE is absorbed hysteretic energy by element during response history. To obtain overall structural damage index, following calculations are made:

$$DI_{Story} = \sum(\lambda_i)_{Component}(DI_i)_{Component} \quad ; \quad (\lambda_i)_{Component} = \left(\frac{Ei}{\sum Ei}\right)_{Component}$$

$$DI_{Overal} = \sum(\lambda_i)_{Story}(DI_i)_{Story} \quad ; \quad (\lambda_i)_{Story} = \left(\frac{Ei}{\sum Ei}\right)_{Story} \tag{17}$$

5.2.8 Base Shear Index

The evaluation of seismic performance of buildings based on the base shear parameter is the simplest estimation method. Therefore, damage index is defined as an equation.18.

$$DI = \frac{s_{cap}}{s_{req}} \tag{18}$$

In the above relationship, R is the damage index, s_{cap} is the available shear strength and s_{req} the shear force capacity of the earthquake. If the DI property is low (DI<0.5), the structure's strength is not satisfactory, but if this parameter is large (DI > 1), the structure has sufficient safety against the earthquake and for the values between the two upper limits can be improved by Relative safety of the structure [32].

6. Results and Discussion

The status of the steel frames is assessed based on the damage indices presented in accordance with the following definitions in tables 6 and 7; therefore, prioritization of ground motion records/damage indices, in this paper is defined based on more conservative values in terms of more damageability rate.
This research allows the designer to either determine the damage level for a every structure under any other seismic loads with different natures, or dimension a structure for special seismic loads and desired level of damage, or determine the maximum seismic load a designed structure can sustain in order to exhibit a desired level of damage according to FEMA-356 code and HAZUS report.

Table 6. Status of steel frames in terms of local damage indices under the expected earthquakes

Structural Elements		Beam				Column			
Damage Index	Earthquake	2-story	5-story	8-story	12-story	2-story	5-story	8-story	12-story
Banon Failure	Manjil	n-c*	C***	C	c	s-c**	c	c	c
	Tabas	C	s-c			c			c
	Sanfernando	c		s-c		c	s-c	s-c	
	Kobe		c	s-c	s-c	n-c	s-c	s-c	s-c
Ductility	Manjil	n-c	c	C	c	s-c	c	c	c
	Tabas		n-c	n-c	n-c	c	n-c		
	Sanfernando					s-c		n-c	s-c
	Kobe					n-c			n-c
Kinematic	Manjil	n-c	n-c	C	c	c	c	c	c
	Tabas				n-c	n-c	n-c		s-c
	Sanfernando			n-c				n-c	
	Kobe								n-c
Banon Normalized Cumulative Rotation	Manjil	n-c	n-c	C	c	n-c	c	c	c
	Tabas			n-c	n-c		n-c	n-c	n-c
	Sanfernando							c	c
	Kobe							n-c	n-c
Cumulative Plastic Rotation	Manjil	n-c	n-c	C	c	s-c	c	c	c
	Tabas			s-c	s-c	c	s-c	s-c	s-c
	Sanfernando								c
	Kobe			n-c	n-c	n-c	n-c	n-c	n-c
Park-Ang	Manjil	c	s-c	C	c	c	c	c	c
	Tabas			s-c	s-c	c	c	c	s-c
	Sanfernando								c
	Kobe			n-c	n-c	s-c	s-c	n-c	n-c

Table 7. Status of steel frames in terms of global damage indices under the expected earthquakes

Damage Index	Earthquake	2-story	5-story	8-story	12-story
Drift	Manjil	c	c	c	c
	Tabas	c	c	c	c
	Sanfernando	c	s-c	s-c	c
	Kobe	n-c	n-c	n-c	n-c
Maximum Roof Displacement	Manjil	c	c	c	c
	Tabas	s-c	s-c	s-c	s-c
	Sanfernando	s-c	s-c	s-c	s-c
	Kobe	n-c	n-c	n-c	n-c
Roufaiel and Meyer	Manjil	c	c	c	c
	Tabas	c	c	n-c	n-c
	Sanfernando	n-c	n-c	n-c	n-c

	Kobe	n-c	n-c	n-c	n-c
Papadopoulos	Manjil	c	c	c	c
	Tabas	c	c	n-c	n-c
	Sanfernando	s-c	n-c	n-c	n-c
	Kobe	n-c	n-c	n-c	n-c
Rosenblueth	Manjil	c	c	c	c
	Tabas	c	c	c	c
	Sanfernando	c	c	s-c	s-c
	Kobe	n-c	c	n-c	n-c
Ductility	Manjil	c	c	c	c
	Tabas	c	c	c	c
	Sanfernando	s-c	s-c	n-c	s-c
	Kobe	n-c	n-c	n-c	n-c
Sozen	Manjil	c	c	c	c
	Tabas	c	c	n-c	n-c
	Sanfernando	c	s-c	n-c	n-c
	Kobe	n-c	n-c	n-c	n-c
Banon Failure (Weighted Function)	Manjil	c	c	c	c
	Tabas	c	c	c	s-c
	Sanfernando	s-c	c	s-c	s-c
	Kobe	s-c	n-c	n-c	n-c
Interstory Rotation	Manjil	c	c	c	c
	Tabas	c	s-c	c	s-c
	Sanfernando	c	s-c	n-c	s-c
	Kobe	n-c	s-c	n-c	n-c
Hysteretic Energy	Manjil	s-c	s-c	c	c
	Tabas	s-c	s-c	c	c
	Sanfernando	s-c	s-c	s-c	s-c
	Kobe	n-c	s-c	s-c	s-c
Park-Ang	Manjil	c	c	c	c
	Tabas	c	c	c	c
	Sanfernando	s-c	s-c	c	c
	Kobe	s-c	s-c	c	c

*n-c (Non-Critical): It means that the "Damage Index" is less than 25% and the performance level of structure is Immediate Occupancy.
**s-c (Semi-critical or moderate damage): It means that the damage index is between 25% to 50% and the performance level is Life Safety.
***c (Critical) It means that damage index is more than 50% and the damage is extensive as well as the performance level is near collapse.

7. Conclusions

Damage indices consider different aspects of structural response with the objective of producing a quantitative measure of structural damage. Calculation of most damage indexes involves complicated and time consuming computations that are neither economical nor feasible in concurrent structural engineering practice. Then in this paper, In order to investigate on the seismic vulnerability and determine the seismic damage indices such as (local and global) it was tried to model, design and perform non-linear time history analysis for multi-story steel frame structures using the finite element software SeismoStruct V 7.0.6. Each of these indicators can be considered as a benchmark for assessing the damaged buildings. So, structural responses were determined in each case and then the damage indices are compared and prioritized. According to the studies and analyses carried out, the most important findings are presented following in this section:

1) It is estimated that approximate damage story is related to the middle beams/columns considerably are more than the side beams/columns, and in some cases, despite the considerably damage that can be repaired in the middle beams and columns. It has also been observed that the distribution of damage to members (2, 5, and 8-story) is same.

2) The damage rate is based on the Interstory rotation damage index has the same mechanism to Banon failure damage index (weighted index).

3) The damage level strongly is depended on selected ground motion. Prioritization (more conservative damage estimation) of earthquake records in terms of damageability rate are:

1-Manjil (moderate to extensive damages) 2-Tabas and San Fernando (limited damage) 3-Kobe (low or no damage).

4) Prioritization of damage indices based on the criticality and severity of the damage index individually as follows:

Prioritization Local Damage Indices:

1- Banon Failure; 2- Park-Ang; 3- Ductility; 4- Banon Normalized Cumulative Plastic Rotation; 5- Kinematics; 6- Cumulative Plastic Rotation.

Prioritization Story Damage Indices:

1- Drift; 2- Banon Failure (weighted index); 3- Interstory Rotation.

Prioritization Global Damage Indices:

1- Park-Ang; 2- Hysteretic Energy; 3- Rosenblueth; 4- Ductility; 5- Sozen; 6- Papadopoulos; 7- Roufaiel and Meyer; 8- Base Shear.

For example, it means that the Park-Ang global damage index is the most conservative.

5) Park-Ang as a combined index is exact damage indicator for specifying local/global damage.

6) No members of the structure (Beam / Columns) has reached the state of collapse and by increasing the number of stories, the value of damage index usually decreases, which is maybe related to the type of selected ground motion records or conservative design methods for taller buildings.

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REFERENCES

[1] Rajeev, P. and Wijesundara, K. K. Energy-based damage index for concentrically braced steel structure using continuous wavelet transform, Journal of Constructional Steel Research, Vol. 103, pp. 241-250, 2014.

[2] FEMA. FEMA 356–prestandard and commentary for the seismic rehabilitation of buildings. Federal Emergency Management Agency, Washington, DC, 2000.

[3] HAZUS-MH MR1 Advanced Engineering Building Module, Federal Emergency Management Agency, 1999. [44] ASCE/SEI. Minimum design loads for buildings and other structures, ASCE 7-05. American Society of Civil Engineers/Structural Engineering Institute, Reston, VA; 2010.

[4] Whitman, R.V.; Reed, J.W. and Hong, S.T. Earthquake Damage Probability Matrices, Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, Italy, 1973.

[5] T. Shiga, A. Shibata and T. Takahashi, Earthquake damage and wall index of reinforced concrete buildings, Proc.ohoku District Symp., Architectural Institute of Japan, pp. 29Ð32, 1968.

[6] Y.Yang, L.Yang, Empirical Relationship between Damage to Multistory Brick Buildings and Strength of Walls during the Tangshan Earthquake, Proc. 7th World Conf. On Earthquake Engineering, Vol. 6, Istanbul, pp 501-508, 1980.

[7] Yuji Ishiyama, Introduction to Earthquake Engineering and Seismic Codes in the World, 2012.

[8] Kazemi H. Ashtiany M. Azarbakht A., Development of Fragility Curves by using New Spectral Shape Indicators and a Weighted Damage Index: Case Study of the City of Mashhad, Iran, Journal of Earthquake Engineering and Structural Vibration, 2015.

[9] Nakano Y. and Okada T. "Reliability analysis on seismic capacity of existing reinforced concrete buildings in Japan" Journal of Structural and Construction Engineering, AIJ; 406, 37-43, 1974.

[10] Stephens, J. E.; Yao, J. T. P. Damage Assessment Using Response Measurements. J. Struct. Eng. ASCE. 113, 4, pp. 787-801, 1987.

[11] Bertero, V. and Bresler, Y. Improved Shaking and Damage Parameters for Post-Earthquake Applications, Proceedings of the SMIP01 Seminar on Utilization on Strong Motion Data, Los Angeles, California, September 12, p. 1-22, 2001.

[12] Banon H., Veneziano D., Seismic Damage in Reinforced Concrete Frames. Earthquake Engng. a. Struct. Dyn. 10, 179-193, 1982.

[13] Krawinkler, H. and Zohrei, M. Cumulative Damage in Steel Structures Subjected to Earthquake Ground Motions, Compute. Struct. 16 (1-4), 531-54, 1983.

[14] Park, Y.J, Reinhorn, A.M., and Kunnath, S.K., Inelastic Damage Analysis of Frame Shear Wall Structure, Technical Report NCEER 87-0008, 1987.

[15] Park Y.J., Ang A.H.-S., Mechanistic Seismic Damage Model for Reinforced Concrete.J. of Struct. Engng. (ASCE), 111, 3, 722-739, 1985.

[16] Roufaiel M.S.L., Meyer C., Analytical Modelling of Hysteretic Behavior of RC Frames. J. of Struct. Engng. 113, 3, 429-444, 1987.

[17] G.H.Powell and R.Allahabadi, Seismic damage prediction by deterministic methods: concept and procedures, Earthquake Engineering and Structural Dynamics, 16, 1998.

[18] Corteza,Y. "Correlation of Building Damage with Indices of Seismic Ground Motion Intensity during the 1999 Chi-Chi, Taiwan Earthquake", International Workshop on annual Commemoration of Chi-Chi Earthquake Taipei, Taiwan, R. O. C., September 18-20, 2000.

[19] Bracci J.M., Reinhorn A.M., Mander J.B., Kunnath S.K., Deterministic Model for Seismic Damage Evaluation of Reinforced Concrete Structures. Techno. Rep.NCEER-89-0033, State Univ. of New York, Buffalo, 1989.

[20] Krawinkler H., Nasser A.A., Seismic Design based on Ductility and Cumulative Damage Demands and Capacities, Non-linear Seismic Analysis and Design Reinforced Concrete Buildings, Edited by: Fajfar P., Krawinkler H., Elsevier Applied Science, 1992.

[21] Kevil, Oghlo, Classifications of Structural Types and Damage Patterns of Buildings for Earthquake Field Investigation. Proc. of the 12th World Conf. of Earthquake Engng. Auckland, New Zealand, 2000.

[22] Daali, M.L., and Korol, R.M., Adequate ductility â steel beams under earthquake-type loading Engineering Structures, Vol. 18, NO.2 , pp. 179-1 89, 1996.

[23] A.Ghobarah, H.Abou-elfath and A.Biddah, Response-based damage assessment of structures, Earthquake Engineering and Structural Dynamics, 28, 1999.

[24] P.S.Skjaerbaek, S.R.K.Nielsen, P.H.Kirkegaard and A.S.Cakmak, "Damage localization and quantification of earthquake excited RC- frames", Earthquake Engineering and Structural Dynamics, 27, 1998.

[25] Miyakoshi, j. and Hayashi, Y. Correlation of Building Damage with Indices of Seismic Ground Motion Intensity during the 1999 Chi-Chi, Taiwan Earthquake, International Workshop on annual Commemoration of Chi-Chi Earthquake Taipei, Taiwan, R. O. C., September 18-20, 2000.

[26] Mikami, T. and Imura, H, Demand Spectra of Yield Strength and Ductility Factor to Satisfy the Required Seismic Performance Objectives, Proceeding of JSCE, No. 689/1-57, p. 333-342, 2001.

[27] Papadopoulos, P.; Mitropoulos, E.; and Athanatopoulou, A. Failure Indices for R/C Building Structures”, 12th European Conference on Earthquake Engineering. Paper Reference 616, Elsevier Science Ltd, 2002.

[28] Barghi, M., and Abasnia, R. Augury of RC columns destruction type in cyclic lateral load. Proc, 7th Int. Conf. on Civil Engng, Tehran, Iran, 2004.

[29] Kianfar, A., Estekanchi, H. and Vafaei, A. A study of damage indices performance in seismic analysis of steel frames" (in Farsi), Proceedings of the 2nd National Congress on Civil Engineering, IUST, Iran, 1025, pp.1-8 2005.

[30] M.Gerami and F. Daneshjoo, Study the Function of Deformation Energy Parametric Assessing Seismic Damages in Steel Frames, Journal of Earthquake Engineering, 2010

[31] Jeong, SH. and Elnashai, AS. New three-dimensional damage index for RC buildings with planar irregularities, Journal of Structural Engineering, 132(9), 1482-1490, 2006.

[32] Ganjavi B, Vaseghi Amiri J, Ghodrati Amiri G, Sarvghad moghadam A R. Distribution of damage, absorption and loss of seismic energy in moment resisting RC frames (MDOF). Proceedings of the 2nd national congress on civil engineering, Iran University of Science and Technology, 2014.

[33] Kabir Sadeghi, “Energy based structural damage index based on non-linear numerical simulation of structures subjected to oriented lateral cyclic loading”, International Journal of Civil Engineering, Vol. 9, No. 3, pp 155-164, Accession Number: WOS:000295205800002, IDS Number:824MK, ISSN: 1735-0522, Sept. 2011.

[34] Vui Van Cao, Hamid Reza Raonagh. Correlation between seismic parameters of far-fault motions and damage indices of low-rise reinforced concrete frames, soil dynamic and earthquake engineering 66, 102-112, 2014.

[35] Jeong G.D et al., the Effect of Earthquake Duration on the Damage of Structures. Earthquake Engng. a. Struct. Dyn. 16, 1201-1211, 1988.

[36] Morik et al, A. and Simon, R., Use of constant cumulative ductility spectra for performance-based seismic design of ductile frames. 13th U.S. National Conference on Earthquake Engineering, 2014.
[37] Shabani Afrafli, Somayeh and Gholamreza Abdollahzadeh, The effect of viscoelastic dampers on the reduction of seismic damage of steel frames under distant disturbances, Second international conference on geotechnical and seismic engineering, Tabriz, Knowledge base of the Azerbaijani earthquake stabilization Tabriz University, 1394.
[38] AR Emami, AM Halabian, Spatial distribution of ductility demand and damage index in 3D RC frame structures considering directionality effects, The Structural Design of Tall and Special Buildings 24 (16), 941-961, 2015.
[39] Abbasi, Shahram and Rahman Mirzai, Seismic Evaluation of Concrete Buildings in Comparison with Failure Characteristics and Fragility Curve Design, Second International Conference on New Research Findings in Civil Engineering, Architecture and Urban Management, Tehran, International Confederation of World Inventors IFIA, Applied University of Applied Sciences, 1395.
[40] H. Mirzaaghabeik, H.R. Vosoughifar, Comparison between quality and quantity seismic damage index for LSF systems, Journal Volume, Pages 497–510, 2016.
[41] Zameeruddin. Mohd, Saleemuddin, Mohd and K. Sangle, Seismic Damage Assessment of Reinforced Concrete Structure using Non-linear Static Analyses, KSCE Journal of Civil Engineering, 2016.
[42] Liu.Z, Chen. K, Zhao. P and Chen. W, Correlation between global damage index and local damage indexes for the seismic performance assessment of framework, Tumu Jianzhu yu Huanjing Gongcheng/Journal of Civil, Architectural and Environmental Engineering 39(3):92-98, 2017.
[43] ETABS (2013), Linear and Non-linear Static and Dynamic Analysis and Design of Three Dimensional Structures, Computers and Structures, Inc. Berkeley, California.
[44] Zheng He, Yichao Xu, Correlation between global damage and local damage of RC frame structures under strong earthquakes: Correlation Between Global Damage and Local Damage of RC Frames, Structural Control and Health Monitoring 24(3), 2016.
[45] AISC 360-05, Specification for Structural Steel Buildings. American Institute of Steel Construction, Inc., Chicago, IL, 2005.
[46] Kumar M, Stafford PJ, Elghazouli AY. Influence of ground motion characteristics on drift demands in steel moment frames designed to Eurocode 8. Eng. Struct 2013; 52:502–517.
[47] SeismoSoft's (2016), SeismoStruct 7.0.6, a computer program for static and dynamic non-linear analysis of framed structures.
[48] Menegotto M. and Pinto P.E, Method of analysis for cyclically loaded RC plane frames including changes in geometry and non-elastic behavior of elements under combined normal force and bending, International Association for Bridge and Structural Engineering, 15-22. Zurich, Switzerland, 1973.
[49] Next Generation Attenuation of Ground Motions (Nga) Project, 2006.
[50] Newmark N.M., Rosenblueth E., Fundamentals of Earthquake Engineering, Prentice Hall, Englewood Cliffs, NJ, 1974.
[51] Yao J.M, Sozen M.A., Effect of Beam Strength and Stiffness on Dynamic Behavior of Reinforced Concrete Coupled Walls. Univ. of Illinois, Urbana; (Techno. Rep., Civil Engng. Studies, Struct. Res. Series No. 444), 1975.
[52] Prahlad P., Shrikhande M. and Agarwal P. (2007), Performance-based seismic design using normalized hysteretic energy, SEAOC CONVENTION, September 26-29 Squaw Creek, California, USA, 2007; pp 473-487, 2007.