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Investigating the Seismic Performance of Steel Moment-Resisting Frames by Using Damage Index

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ABSTRACT

In the seismic analysis of structures, estimation of amount of damages to the elements is of great importance. In the quantitative method of estimating damages, it is possible to assess damages to the storey and the total structure by introducing damage indices. The results of studies show that damage indices could be used as a suitable basis in assessing the function of structure against back and forth forces of earthquake excitations. These indices are calculated based on functions of deformation, and dissipated energy of elements in frames. In this study, the steel moment-resisting frames with 4, 7, 10, 15, 20, and 25-story and 3 and 5 bays are designed and then, they are modeled in OpenSees software in order to investigate the performance of steel moment-resisting frames under seismic excitations by using damage index. In the following, nonlinear dynamic time history analysis is conducted and the damages of aforementioned frames are calculated quantitatively and the results showed that the behavior of high-rise, mid-rise, and low-rise frames were affected based on their vibration periods and dependent to the nature of the studied earthquakes.

Keywords:

Quantitative method, Damage index, Deformation, Dissipated energy, Nonlinear dynamic time history analysis.

1. Introduction

Standard design approaches based on the notion of the force reduction factor are acknowledged in the seismic design of steel and reinforced concrete (*RC*) structures, even if appropriate in most practical instances, do not result in structures with uniform and rationally defined safety and performance levels [1-4]. As a result, attention to damage indices or damage indicators have been increased in researches. A damage index is a state variable that connects a specific damage condition caused by complex nonlinear deformation, energy dissipation, or low-cycle fatigue to a single point on the monotonic backbone curve. Because a damage index is simply a normalized damage indicator, the two definitions will be viewed as interchangeable in this context. A significant amount of study has been done on the creation of damage indices over the last *20–30* years. In general, structural damage has been classified as either economic or safety/strength-related. Economic damage indices are typically represented as a ratio of repair to replacement costs for a whole structure or a specific structural piece. The loss of structural resistance is usually linked to safety/strength damage indices. In light of previous building failures, the earthquake engineering community understands the need to upgrade present seismic regulations and design approaches. Part of this can be due to obfuscated design approaches like the equivalent static force procedure, which ignores the cyclic load effect that occurs frequently during earthquakes. Cyclic loading, on the other hand, has long been recognized as having a considerable impact on the cumulative damage to structures. This weakness can be addressed by utilizing a more appropriate method of quantifying seismic damage, such as a damage index that incorporates the effects of maximum deformation as well as inelastic energy dissipation. On the one hand, an earthquake is one of the world's most complicated natural events, and predicting the right behavior of structures in earthquakes is extremely difficult. Many efforts have been made and are still being made in this direction, dating back decades. To limit the damage rate after an earthquake, each component of the building must be independently assessed and analyzed. On the other hand, an earthquake as one of the most complex natural phenomena in the world and it is very difficult to predict the proper behaviour of structures in earthquakes. In this regard, many efforts have been made from decades ago and are still ongoing. Each component of building must be individually analysed and evaluated in order to reduce the damage rate during an earthquake. According to the researchers, a numerical number that is a function of structural attributes and external loadings can be used to indicate the state of a damaged member or structure [5]. Many approaches for predicting seismic damage have been developed in recent decades. As a result, significant effort has been made to improve present earthquake resistant design approaches in order to not only avoid collapse in the event of a destructive earthquake, but also to limit damage in the event of a weak earthquake. In addition, the new design philosophy is presented by multi-level probabilistic structural performance criteria over the conventional force strength approach. However, putting all of these new ideas into practice necessitates the creation of a qualitative damage index and measure. In the seismic design of structures, the concepts of local and global damages and also vulnerability of structures play great roles. The damages to structures could be expressed by damage indices. The values of damages indices are usually shown with values ranging from "*zero*" to "*one*". The value "*zero*" shows no damages and "*one*" represents the collapse of element or structure. Also, the values between values zero to one are quantified the damages ranging low to high rate.

Meanwhile, 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 [6].

In this research, the steel moment-resisting frames with *4*, *7*, *10*, *15*, *20*, and *25*-story and *3* and *5* bays are designed and then, they are modelled in *OpenSees* [7] software in order to investigate the performance of steel moment-resisting frames under seismic excitations by using damage index. In the following, nonlinear dynamic time history analysis is conducted and the damages of aforementioned frames are calculated quantitatively.

2. Background of Damage Index

In order to quantify numerically the degree of damage, a damage index is established for the seismic damage assessment of the structures. Damage indices can be used to measure damage and link it to expenses and other repercussions, such as possible risk following an earthquake. As a result, in earthquake-prone areas, damage index can play an essential role in retrofit decision-making and catastrophe preparation.

The structural responses used as damage parameters are divided into three categories:

1. Elements or structures deform due to plastic deformation.

2. Energy dissipation in the elements due to hysteretic behavior: Prior to breakdown, structural elements have a limited capacity to dissipate energy in a cyclic fashion. The amount of energy dissipated is a good estimate of how much harm was done during loading.

3. Changes in the structure's dynamic properties, such as the structure's first natural period.

Damage indices are often standardized to have a value of zero when there is no damage and unity when there is total collapse or failure. A damage parameter, on the other hand, is a quantity that is used to estimate the damage.

Damage indices based on strength are straightforward and do not necessitate a structural response analysis. They are based on geometric properties of structural elements such as the cross-section of beams, columns, braces, and steel and reinforced concrete shear walls, as well as the properties of their materials. These types of damage indices should be calibrated against observed damage utilizing a large real-world database or the results of non-reliance structural analysis.

Shiga et al. (1968) [8] and Yang (1980) [9] were the first researchers to propose a damage index based on strength. A highly extensive analysis is required in the damage evaluation approach based on structural response, and significant information is required to calibrate the results. This method necessitates precise information on structural models, materials, and descriptions of ground motions that are site-compatible [10]. The classification of seismic damage indices in structures is shown in Figure 1.

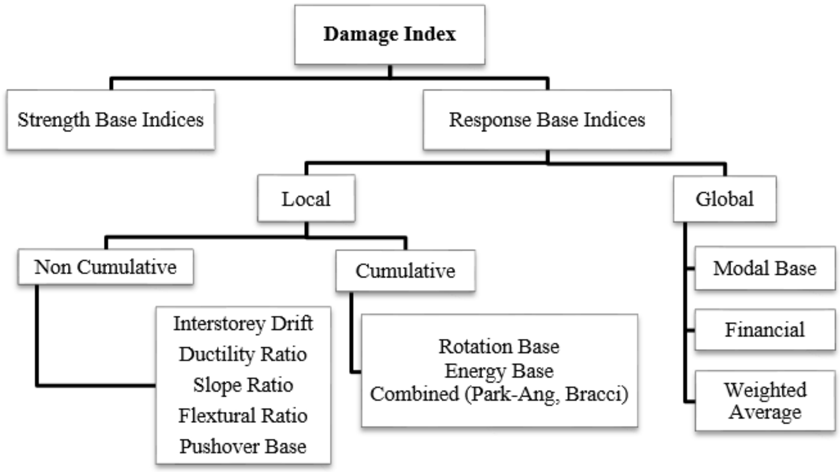


Figure 1. Classification of damage indices [11].

Whitman (1972) expressed the earthquake-induced damage index with the cost of repair to the cost of rebuilding in varying degrees of ground motion [12]. Okada et al. (1974) proposed a method for measuring the seismic safety of RC structures [13]. The damage index based on relative displacement was also introduced by Stephens and Yao (1975) [14]. The definitions of local and global structural damages were established by Bertero and Bresler (1977) [15]. The damage index was presented by Banon et al. (1981) based on the initial stiffness ratio at the maximum displacement of the pushover curve, and the damage model was specified by formability factors in 1982 [16]. Krawinkler et al. (1983) proposed a cumulative damage index that is proportional to the structural performance parameter, plastic deformation, deformation, and the total number of cycle motions [17]. Park et al. (1984) proposed a significant vulnerability evolution. They applied the ductility and energy absorbed by structural members to the damaged members by taking into account more detailed models of non-linear behavior of RC members under oscillatory loads [18]. Park and Ang (1985) proposed a new technique based on the member's maximum deformation and integration with the absorbed energy [19]. Roufaiel and Meyer (1987) assessed the seismicity of steel and RC structures and developed a structural characteristics-based index for the total structure [20]. Powell and Allah Abadi (1988) proposed a method for estimating the damage index that was based on comparing structural capacity during earthquakes [21]. Corteza (1993) articulated the similar relationship, but his was based on the ductility and energy of the hysteresis absorbed in the structure [22]. The global damage index for structures was developed by Bracci et al. (1989) [23]. The failure of structural elements was studied by Krawinkler and Nasser (1992) by using ductility and cumulative damage indices. In this procedure, the corresponding ductility is computed assuming an acceptable amount of damage, and then the strength required to limit the demand ductility to the present capacity is calculated, giving an overview of the structure's behavior [24]. The frequency variation of the initial vibrational mode due to the reduction of stiffness and strength was presented by Kevil Oghlo et al. (1994). They predicted the first vibration mode, local and global damage, by examining the behavior of hysteresis curves [25]. Based on the Park damage index [26], Dali and Korol (1996) presented a damage index. Ghobarah and Abu al-Fattah (1997) established a damage index approach based on structural response and stiffness measurements of various building classes, which is carried out by static load analysis before, during, and after the earthquake [27]. Ghobarah and EI-Attar (1998) proposed a new approach for determining the damage concentration in RC frames. This method obtains an appropriate assessment by requiring the size of the structure response, the ground acceleration, and the first two frequencies of ground motion, resulting in an appropriate assessment [28], particularly in situations where the damage is concentrated at a specific level of the structure. By following the Taiwan earthquake in 1999, John Miyakoshi compared the earthquake damage to building collapses in the Chi-Chi earthquake (1995) and the Kobe earthquake (1995), resulting in a new formula for calculating the building failure index. The schools were chosen based on a collection of empirical data [29]. Mikami and Imura (2000) developed a novel relationship in which the maximum fluctuation and resistivity of steel were considered [30] with the help of Park and Ang (1985) in the elastic range and softness. In a simple but accurate manner, based on plastic joints in columns, Papadopoulos et al. (2004) proposed a criterion for degradation measures that, in addition to the prior methods, was also convenient [31]. The number of 25 RC columns with a specific loading history were analyzed by Abbas Nia and Electric (2004) with the objective of examining and criticizing Park and Ang's damage index [32]. Kianfar, Estekanchi, and Vafaei (2004) used various damage indices to evaluate the performance of 3rd and 7th floor frames [33]. Jeong and Elnashai (2006) proposed the building of fragility curves for irregular structures in a suggestion that included a multidimensional relation for the locomotive damage index [34]. Barghi and Rajabi (2010) investigated the creation of a Park-Ang damage model on concrete columns with flexural and cyclic loads, using experimental results from 95 columns [35]. In a cyclic loading model [36], Sadeghi (2011) suggested a simple and exact damage index for measuring structural damage. The association between seismic parameters of far faults ground motions and the damage index of short RC frames was presented by Vui Van Cao et al. (2014) [37]. For measuring the failure of symmetric structures in the plan, Morik et al. (2014) presented a combined damage index [38]. In addition, Rajeev et al. (2014) presented a damage index for Concentric Braced Frame (CBF) constructions based on the amount of absorbed energy [39]. The seismic sensitivity of 7 and 10 story reinforced concrete frame structures was compared to the damage indices of the class interfaces and the pulp length of the joints in the fragility curve [40] by Abbasi and Mirzaei (2016). By using the Papadopoulos damage index, Mirzaaghabeik et al. (2016) objectively and qualitatively evaluated and compared lightweight steel frame constructions considering the interaction of soil-structure [41]. Zameeruddin et al. (2017) used nonlinear static analysis to assess seismic damage indices of RC frame structures [42]. Suraj et al. (2020) presented drift limits for RC frame staging in raised water tanks for various seismic damage states. Using the Park and Ang damage index, several damage states of the elevated water tank were determined. The Park and Ang damage index incorporates both pushover and incremental dynamic analysis results. Twelve kinds of elevated water tanks were created with different staging heights and tank capacities in mind. The suite of twelve genuine earthquake ground motions was used to perform incremental dynamic analysis. Limiting drift values for each damage stage are proposed based on the regression analysis between damage indexes and drift [43].Zhang (2021) proposed a method for calculating structural total damage by integrating two damage components from two vertically opposed directions. A new seismic failure checking approach for steel reinforced concrete (SRC) frame structures is suggested based on the damage index. Damage limitations in this approach correspond to three design requirements of Chinese codes, and this damage-based method seeks to check the failure status of SRC frame structures [44]. Hosseini et al. (2022) investigated three distinct damage indices for detecting nonlinear damages in two nearby RC structures when pounding effects were taken into account. 2, 4, and 8-story benchmark RC Moment Resisting Frames with 60%, 75%, and 100 % minimum separation distance and no in-between separation gap were chosen for this purpose. As a result, nonlinear damages can be recognized using damage indices for a certain seismic intensity [45].

3. Modelling Procedure

In order to assess seismic vulnerability and determine damage indices, requires modeling and analysis on structures, if possible, experimental studies and comparison of results is necessary. The results of theoretical modeling is useful, but since laboratory studies are costly, in order to study and compare the amount of damage to members and stories in steel buildings with a moment-resisting frame system and the number of stories (4, 7, 10, 15, 20, and 25) and regular and simple geometry of plan has been used to be able to accurately evaluate the deformation and energy parameters in the studied damage indices. The plan of the studied buildings, the length of the bays and the height of each stories have been selected based on the paper of Kumar et al. [46] shown in Fig 2. In this study, in order to perform nonlinear dynamic time history analysis and extract the structural damage index, axis 2 of mentioned frames was selected from the steel buildings. The position of the frame of the steel structures is specified in Fig 3 and the configuration of the frames is indicated in Fig 4. In this study, frames 4, 7, 10, 15, 20, and 25 stories, 3 and 5 bays were designed. In those frames, the height of story is 3 meters and the length of bays is 4.5 meters. Type of materials were construction steel ST-37 with yield strength of 240 MPa and module of elasticity of 200 GPa were considered. Based on Tab 1, cross sections used for the beams of frames were HEB and for columns, they are BOX. The dead load of stories for all structures was 300 kg/m2, the load dead of the roof floor was 250 kg/m2. In the following, the live load of stories are 200 kg/m2 and the live load of roof floor was 150 kg/m2. In order to calculate the lateral load, 2800 standard, 4th edition was used [47]. The frames designed with the spectrum of this code underwent Tabas, Manjil, El Centro and Borrego Mountain earthquakes and the values of damage indices of different frames were obtained. Based on Fig 5, the maximum horizontal acceleration values of Tabas and Manjil earthquakes were 0.82 g and 0.56 g, and the values of El Centro and Borrego Mountain earthquakes were 0.31 g and 0.06 g; respectively. The earthquakes were considered in a way so the difference in damages and functions of various damage indices in frame stories would be evaluated and compared. To perform nonlinear dynamic analysis, the OpenSees software was used. The strain hardening was considered 3% [48-52]. To perform non-linear dynamic analysis in Opensees program, each elements is divided into 10 parts and in these parts in the three points of upper and lower corners and middle of cross sections, the stresses and strains were extracted and the loss cycle energy of each member is calculated from it.

Table 1. Cross sections of studied frames.

No. story	Beam sections	Column sections
4	HE240B & HE220B	BOX200X200X20
7	HE280B & HE220B	BOX200X200X25 & BOX200X200X16
10	HE320B & HE300B & HE280 &HE260B	BOX280X280X35 & BOX240X240X40
15	HE400B & HE360B & HE340 &HE260B	BOX300X300X35 & BOX300X300X20 & BOX250X250X20
20	HE500B & HE450B & HE400B & HE360B & HE340 &HE280B & HE240B	BOX400X400X40 & BOX350X350X35 & BOX320X320X20 & BOX300X300X20 & BOX280X280X20 & BOX260X260X16
25	HE600B & HE550B & HE5000B & HE400B & HE360 & HE320B & HE300B & HE260B	BOX500X500X40 & BOX450X450X35 & BOX420X420X20 & BOX400X400X20 & BOX350X350X20 & BOX300X300X16 & BOX280X280X16

To define the nonlinear behavior, Fiber Element (wide distribution of plasticity throughout the member) has been used to model the frame members (beam-column) and the nonlinear behavior of steel frame members based on Steel01 is shown in Fig 6. Is. It should be noted that fiber elements are a model that considers nonlinear behavior in a diffuse manner and has been able to show nonlinear properties in steel elements more clearly. The number of fiber sections is 200 and the number of Gaussian integrations along the beam-column elements is 5. In nonlinear dynamic analysis, materials are allowed to enter the realm of nonlinear behavior, resulting in large deformations and dissipation of energy due to material, cracking, and failure. The damping during the analysis can change from one step to the next. But these matrices are constant during each time step and the response of the model under earthquake acceleration is calculated numerically by each time step. In order to perform nonlinear dynamic analysis in OpenSees program, each element is divided into 10 parts and in these parts at 3 points in the upper, lower and middle corners of the section, the corresponding stresses and strains Extracted and from it the lost cyclic energy of each member is calculated.

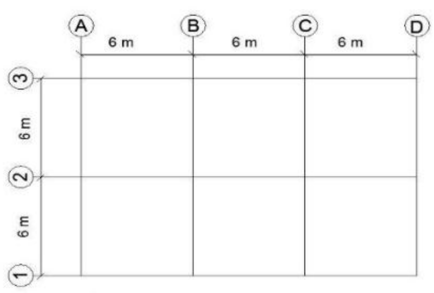


Figure 2. The studied plan of mentioned buildings [46].

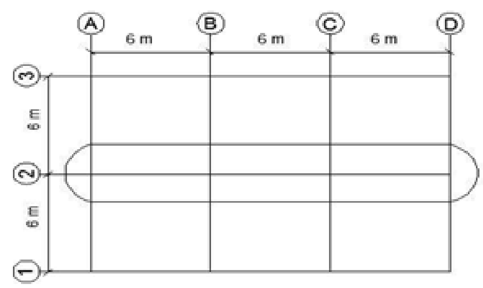
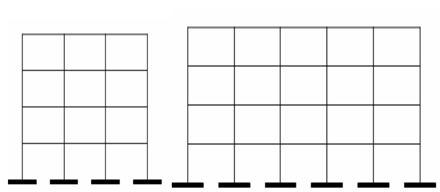
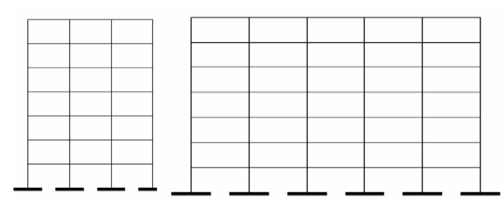


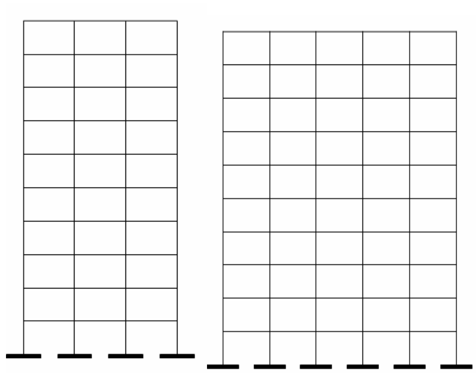
Figure 3. The studied 2D frame in plan [46].



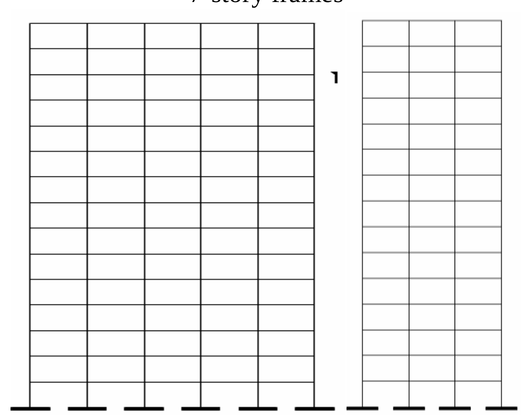
4-story frames



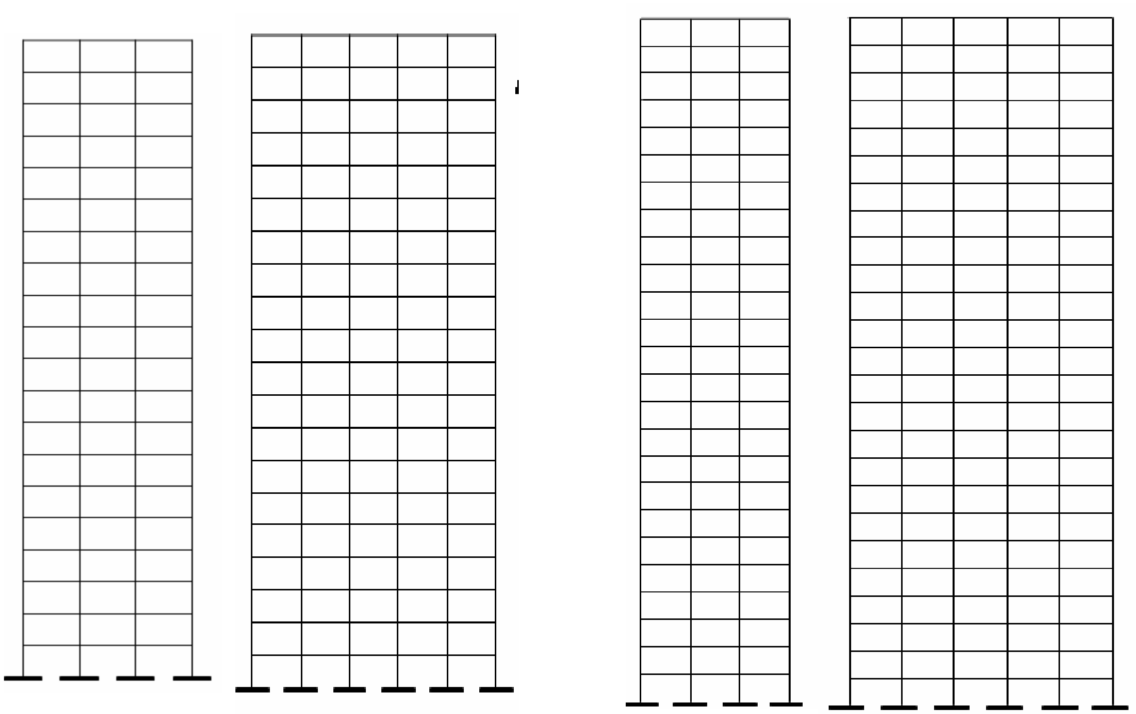
7-story frames



10-story frames



15-story frames



20-story frames

25-story frames

Figure 4. The configuration of the studied frames.

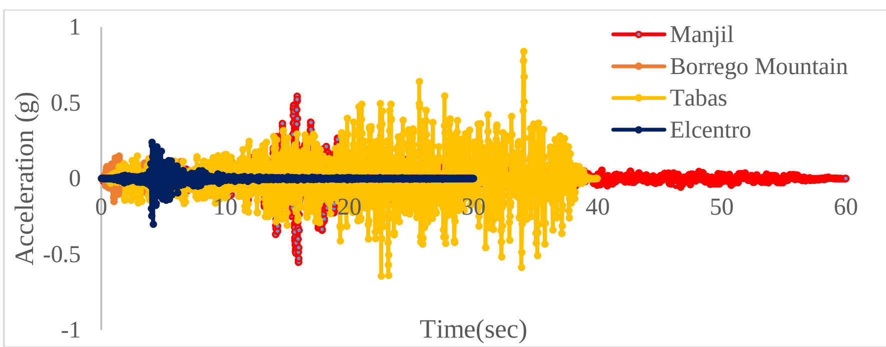


Figure 5. The acceleration-time curve of the studied earthquakes**.**

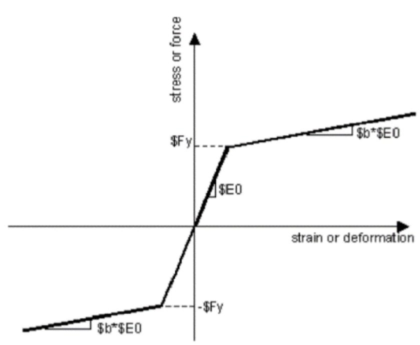


Figure 6. The behavior model of used steel [48-52].

4. The Studied Damage Index

Indices that indicate local damage to a member or total building under earthquake loading are described. In most cases, these indices are dimensionless parameters that take the value of zero for the state without damage and the value of one for the collapse of the structure, while the values between zero and one indicate different degrees of failure. Most local damage indices are naturally cumulative, reflecting the dependence of damage on the amplitude and number of load fluctuations.

4.1. Deformation Index Dependent on Deformation

The simplest definition of a failure function, according to Equation (1), is the ductility damage index [5]. The ductility damage index does not take into account the amount of energy dissipation by the elements and only uses the maximum deformation to express the failure rate. In this equation, DI is the ductility damage index, θm is the maximum rotation of the end of the member during an earthquake, and θu is the final rotation of the element section. Values greater than one of this index indicate element failure.

$$DI_{Ductility} = \frac{\theta_m}{\theta_u} \tag{1}$$

4.2. Energy Damage Index

The energy damage index, unlike the ductility damage index, only takes into account the energy dissipation by the elements and does not take into account their deformation. One of the advantages of indices is the effect of earthquake duration on the behavior of the structure and the amount of damage is calculated cumulatively. The energy damage index is defined according to Equation (2) [53]. In EH equations, the energy dissipated by the elements and the My is the yield point of the member.

$$DI_{Energy} = \frac{\mathrm{EH}/M_y\theta_y}{(\theta_u/\theta_y \ - 1)} \tag{2}$$

4.3. Combined Damage Index

Indices that take into account the effects of deformation and the dissipated energy of the elements together can be more reliable. For this purpose, the damage index of Park-Ang is defined according to Equation (3). This index is widely used in studies to evaluate the failure rate of concrete beam elements and has also been used in the case of steel elements [18]. The numerical value of β is experimentally defined as 0.15. Values greater than one indicate the heavy failure and collapse of an element or a structure [19].

$$DI_{Park-Ang} = \frac{\theta_m}{\theta_u} + \beta \frac{\mathrm{EH}}{M_y\ \theta_u} \tag{3}$$

5. Results and Discussions

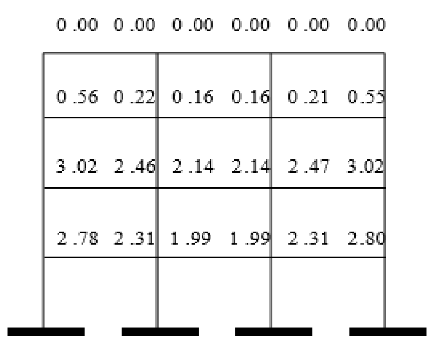
To study and analyze the total damage of structures, the frames subject of study have been divided into three groups:

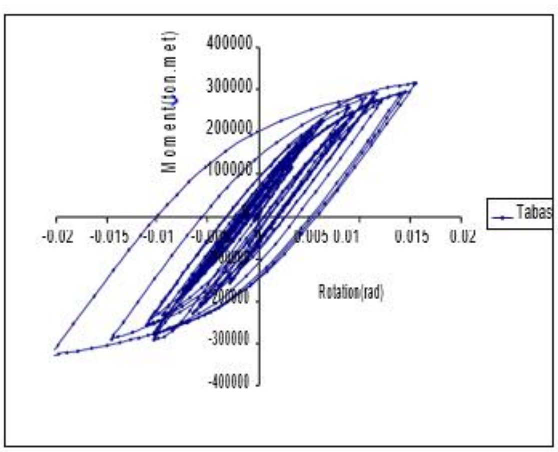
Group I: Low-rise frames including 4 and 7-story frames.

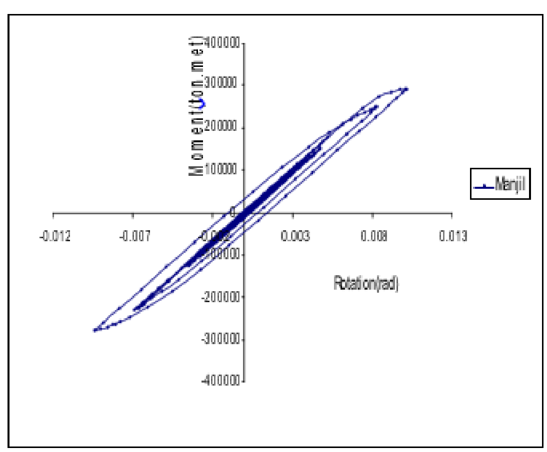
Group II: Mid-rise frames including 5 and 10-story frames.

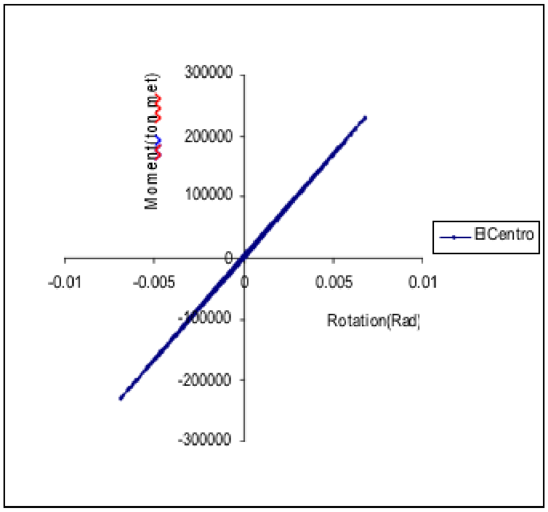
Group III: High-rise frames including 20 and 25-story frames.

According to Figs 7 to 12, the changes in amount of total damage indices of the studied frames are shown based on their vibration periods. Based on the aforementioned figures, it is possible to compare the damages of the studied frames under different earthquakes and analyzing the results. As it could be seen, in Tabas earthquake case, the mid-rise frames suffered greatest damages and the story collapses occurred. The 7-story mid-rise frames; faced collapse of stories. The least damage index of Park-Ang under this earthquake was recorded in the 4-story low-rise frame. The high-rise frames; faced "intensive" damages (according to the definitions of the ranges of this index) and the value of this index for the high-rise 20-story frame was 0.9 and for the 25-story, it was 0.85. As it is noticeable, the damages were higher than other earthquakes under Tabas earthquake and based on Park-Ang damages index. In Manjil earthquake, the greatest damages were occurred in 15-story mid-rise and 20-story high-rise frames. The least damages were seen in low-rise frames. Even amount of damages in the 7-story frame was little less than damages to the 4-story frame. In El Centro earthquake, as the diagrams show, the function of this earthquake was similar to that of Manjil earthquake. The Borrego Mountain earthquake caused fewer damages than other earthquakes. The amount of this index under this earthquake was larger in the high-rise frames.









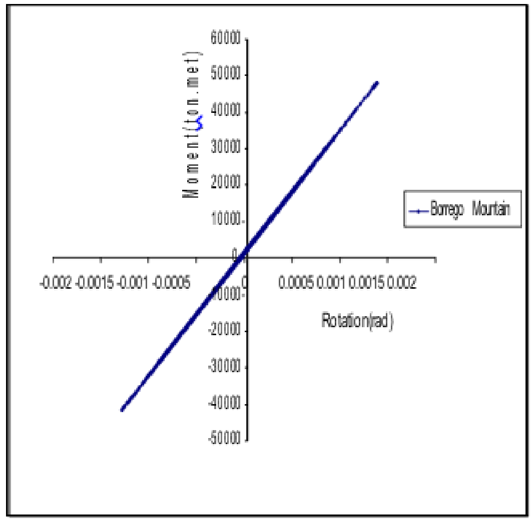
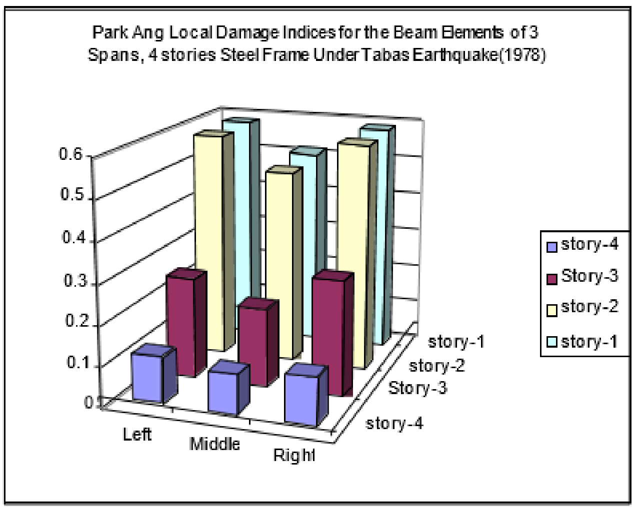


Figure 7. Moment-Rotation hysteresis curve of the left end of beam in left bay of the first story of 4 story- 3 bay frame under the studied earthquakes**.**



Story	Left	Middle	Right
Story1	Intensive	Intensive	Intensive
Story2	Intensive	Intensive	Intensive
Story3	Average	Slight	Average
Story4	Slight	No-Damage	Slight

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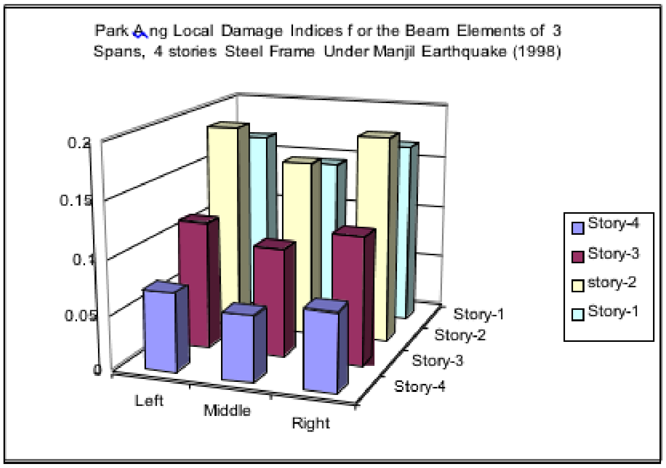
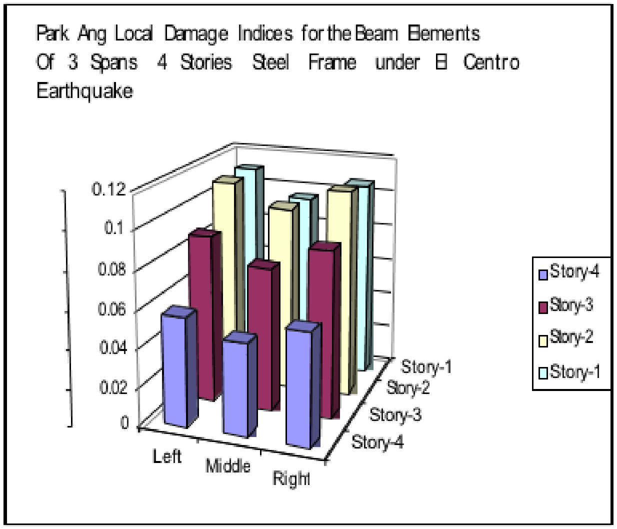


Story	Left	Middle	Right
Story1	Intensive	Intensive	Intensive
Story2	Intensive	Intensive	Intensive
Story3	Average	Slight	Average
Story4	Slight	No-Damage	Slight



Story	Left	Middle	Right
Story1	Intensive	Intensive	Intensive
Story2	Intensive	Intensive	Intensive
Story3	Average	Slight	Average
Story4	Slight	No-Damage	Slight

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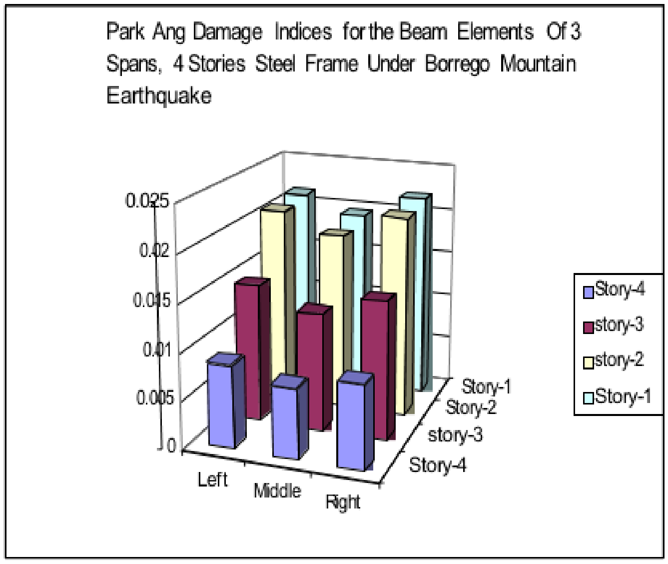
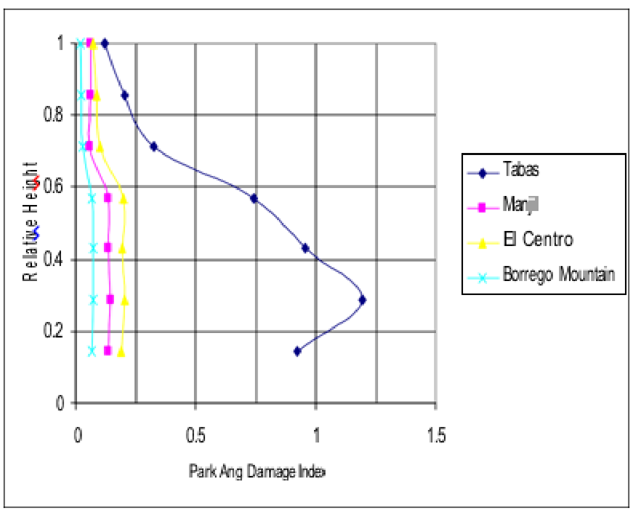
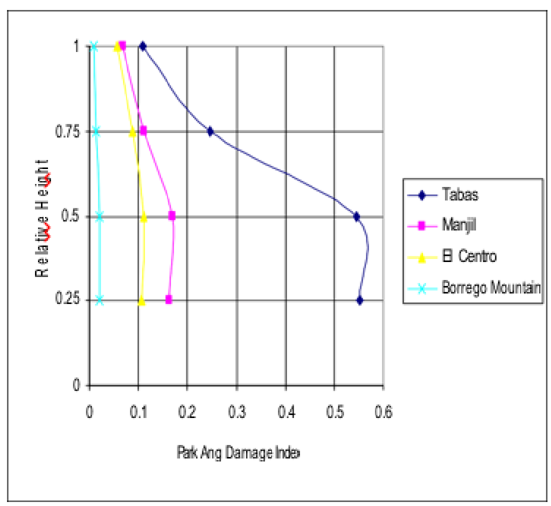


Story	Left	Middle	Right
Story1	Intensive	Intensive	Intensive
Story2	Intensive	Intensive	Intensive
Story3	Average	Slight	Average
Story4	Slight	No-Damage	Slight

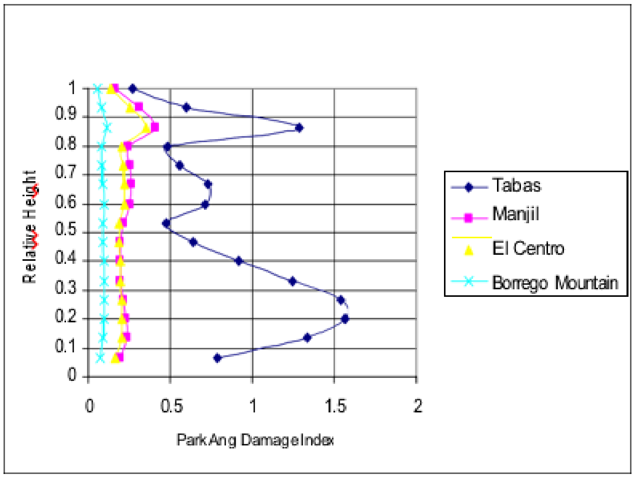
Figure 8. Story Park Ang damage indices under the studied earthquakes for 4-story frame with 3-bay.



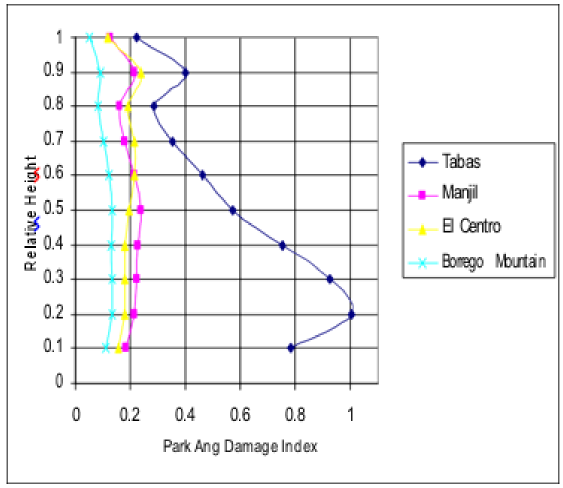
4-story frame



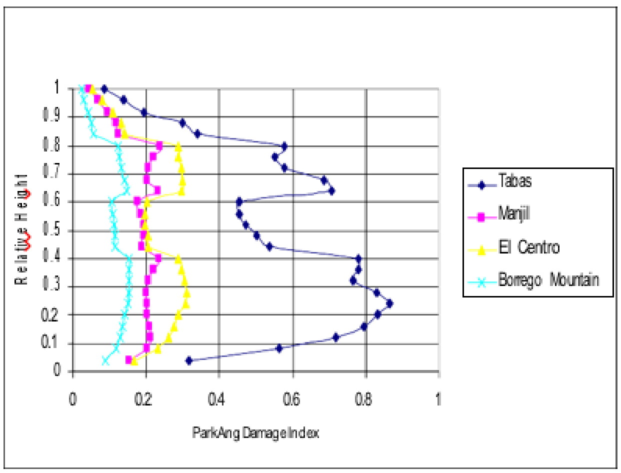
7-story frame



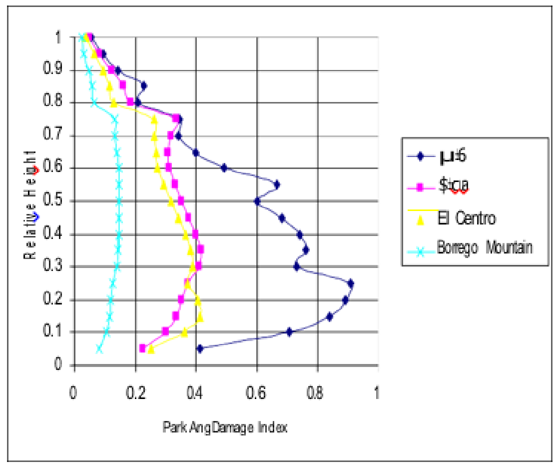
10-story frame



15-story frame

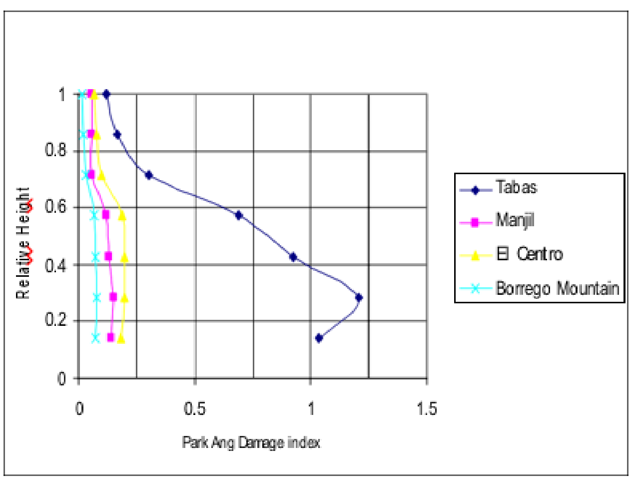


20-story frame

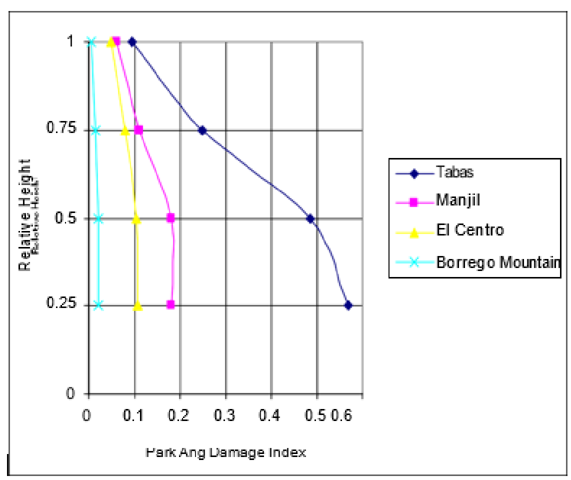


25-story frame

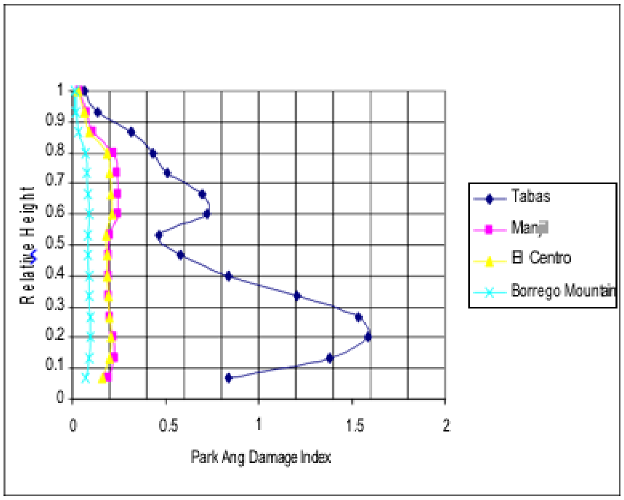
Figure 9. Story Park Ang damage indices under the studied earthquakes for frames with 3-bay.



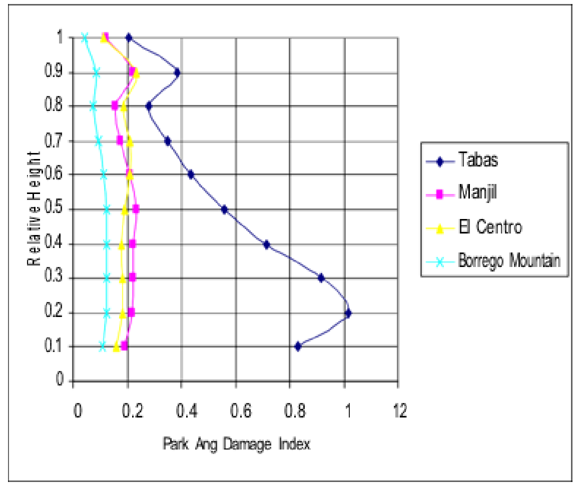
4-story frame



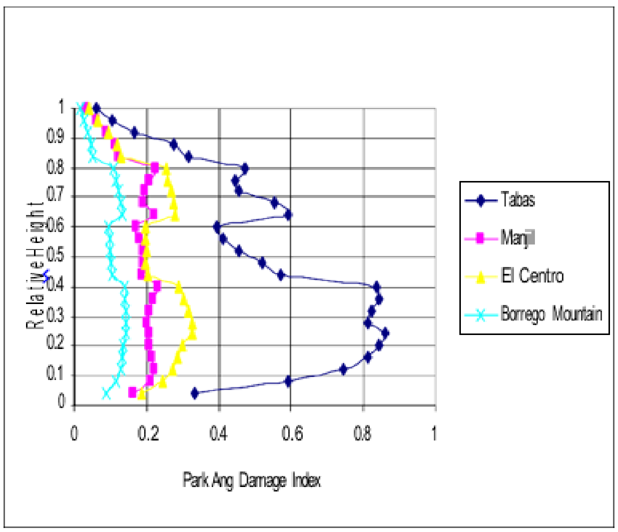
7-story frame



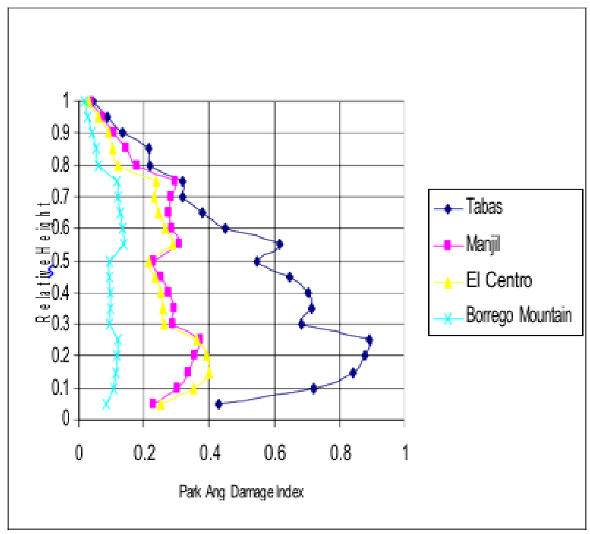
10-story frame



15-story frame



20-story frame



25-story frame

Figure 10. Story Park Ang damage indices under the studied earthquakes for frames with 5-bay.

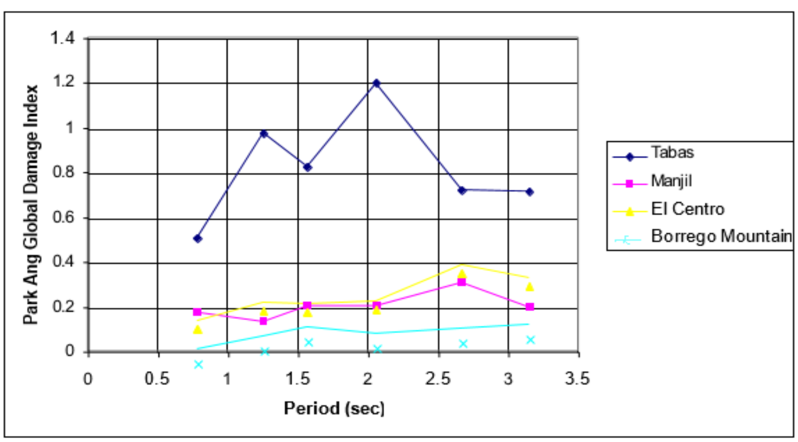


Figure 11. Global Park Ang damage indices under the studied earthquakes for frames with 3-bay.

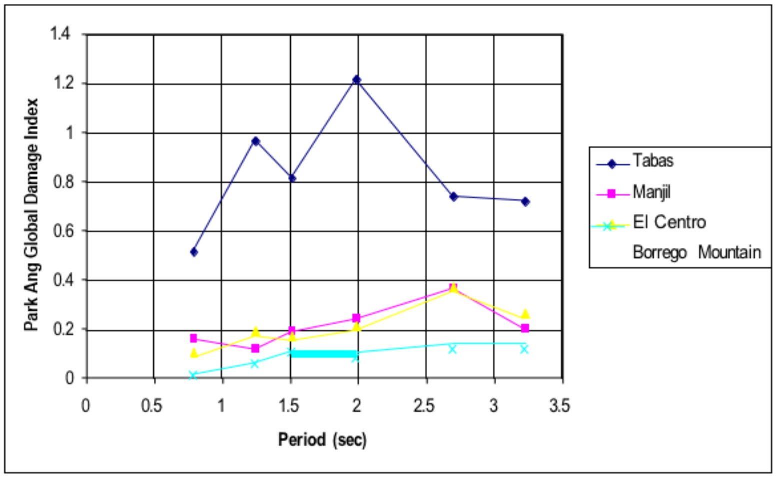


Figure 12. Global Park Ang damage indices under the studied earthquakes for frames with 5-bay.

6. Conclusions

In order to investigate the seismic vulnerability and determine the seismic damage indices such as (local and global) it was tried to model, design and nonlinear dynamic time history analysis for multi-story steel frame structures using the OpenSees software. Park-Ang damage index can be considered as a benchmark for assessing the damaged buildings. So, structural responses were determined in each case. According to the studies and analyses carried out, the most important findings are presented in this section:

-The effect of number of bay in frames were similar and no significant differences were seen.

-With respect to the results obtained in the studies, largest damages were seen under Tabas earthquake and the least damages were noticed under Borrego Mountain earthquake.

-Due to consider the both factors of deformation and energy in estimating amount of damage and due to the significance of specific ranges in quantitative interpretation of amount of damage, Park-Ang damage index is more likely to express the realities.

-Under Tabas earthquake, the greatest damages were noticed in the frames of 2 sec vibration period owing to the mid-rise 15-story frames. The least damages were in low-rise 4-story frame with 0.7 sec vibration period. Under Manjil and El Centro earthquakes, the largest damages were in the 15-story mid-rise and 20-story high-rise frames with 2 to 2.7 sec vibration periods.

-Amount of damages in frames was greater in their first 50% of height than damages in the top parts of those frames.

-The possibility of collapse in story was observed under Tabas earthquake. The collapse was mostly in the 2nd story in the 7-story frame; the 2nd in the 10-story frame, 2nd to 5th and in 13th story in the 15-story 3-bay frames and 2nd to 5th in the 15-story, 5-bay frames that collapsed under Tabas earthquake.

-In studying the amount of damages in the whole structure, the mid-rise frames, 15-story frames with 2 sec vibration period under Tabas earthquake suffered greatest damages.

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