Computers and Concrete, *Vol. 21, No. 1 (2018) 55-66*
DOI: https://doi.org/10.12989/cac.2018.21.1.055

Effects of consecutive earthquakes on increased damage and response of reinforced concrete structures

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(Received January 7, 2015, Revised May 31, 2017, Accepted October 11, 2017)

Abstract. A large main shock may consist of numerous aftershocks with a short period. The aftershocks induced by a large main shock can cause the collapse of a structure that has been already damaged by the preceding main shock. These aftershocks are important factors in structural damages. Furthermore, despite what is often assumed in seismic design codes, earthquakes do not usually occur as a single event, but as a series of strong aftershocks and even fore shocks. For this reason, this study investigates the effect and potential of consecutive earthquakes on the response and behavior of concrete structures. At first, six moment resisting concrete frames with 3, 5, 7, 10, 12 and 15 stories are designed and analyzed under two different records with seismic sequences from real and artificial cases. The damage states of the model frames were then measured by the Park and Ang's damage index. From the results of this investigation, it is observed that the sequences of ground motions can almost double the accumulated damage and increased response of structures. Therefore, it is certainly insufficient to ignore this effect in the design procedure of structures. Also, the use of artificial seismic sequences as design earthquake can lead to non-conservative prediction of behavior and damage of structures under real seismic sequences.

Keywords: seismic sequence; damage index; cumulative damage; reinforced concrete structures

1. Introduction

Ground motion sequences separated by the short time intervals have occurred in several parts of the world, including Japan, Mexico, Turkey, Italy, Peru and California. These records have the potential to cause additional cumulative damage to structures due to accumulation of inelastic deformations from all sequences before any repair is possible (Moustafa and Takewaki 2012). Moreover, the low-frequency content in the secondary sequences may cause resonance in the lower modes of the damaged structure leading to further damage. Multiple earthquake sequences may not be an important issue in dynamic analysis of linear structures, since linear structures return to their initial equilibrium position by the end of each sequence. But such ground motions can create severe damage in inelastic structures (Moustafa and Takewaki 2010). For this reason, most researchers are interested in studying about the effect of seismic sequence of dynamic behavior of structures.

To the author's knowledge, the first pioneering analytical study of nonlinear Single- Degree- Of Freedom (SDOF) systems subjected to main shock- aftershock acceleration time histories recorded during the 1972 Managua earthquake were performed by Mahin (1980). The response of structures subjected to main shock-aftershock earthquake ground motion sequences has gained the attention of the earthquake engineering community recently, since strong aftershocks might be triggered after the main shock. For example, after the 1994 Northridge earthquake that affected the Los Angeles Area in California, an aftershock was felt approximately one minute later (Dreger 1997). Therefore, in recent years, social and economic considerations have necessitated that more than one performance criterion and also more than one level of earthquake intensity to be used. Finally, some researchers tried to examine the effect of multiple earthquakes on the response of SDOF and MDOF structures by introducing artificial models for repeated acceleration sequences. For example, Faisal *et al*. (2013) focused on the influence of repeated earthquakes on the maximum story ductility demands of three-dimensional inelastic concrete frames.

Due to lack of real seismic sequence records, only artificial sequences were used. These sequences have been generated by a rational combination of real single events (Faisal *et al*. 2013). The assembly method is obtained from the study of Hatzigeorgiou and Beskos (2009). Some of researchers as Erochko *et al*. (2011), Li and Elliwood (2007), Fragiacomo *et al*. (2004), Amadio *et al*. (2003) proposed "Back to back" or "Repeated" approach which consists of repeating the real main shock, at scaled or identical amplitude, as an artificial aftershock. This assumes that the ground motion features such as frequency content and strong motion duration of the main shock and aftershock(s) are the same.

In these years, others have focused their attention in "randomized" approach, such as Hatzigeorgiou and Liolos

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http://www.techno-press.org/?journal=cac&subpage=8
ISSN: 1598-8198 (Print), 1598-818X (Online)

(2010), Hatzigeorgiou (2010), Luco *et al*. (2004). This approach consists of the ensemble a set of real main shocks, and generating artificial sequences by selecting a main shock and simulating the remaining aftershocks by repeating the main shock wave format repeatedly, at reduced or identical amplitude, with no change in spectral content as an artificial aftershock (Hatzigeorgiou *et al*. 2010). On the other hand, main shock acceleration time histories randomly selected from a ground motion catalog are employed as corresponding aftershocks, scaled or not scaled. In one of the recent studies, researchers have tried to gain further understanding on the effects of soft-soil seismic sequences on the seismic response of RC framed-buildings. This investigation employed artificial sequences by “Back to back” and “Randomize” approach, because only two real sequences were gathered (Garcia *et al*. 2014). Since the information about consecutive earthquakes is currently limited, Moustafa and Takewaki (2011) proposed a simple stochastic model for representing repeated acceleration sequences. So that the ground acceleration was represented as a stationary Gaussian random process modulated by an envelope function of repeated characters. In this model, frequency content for the individual sequences of the same record could be significantly different (Moustafa and Takewaki 2011).

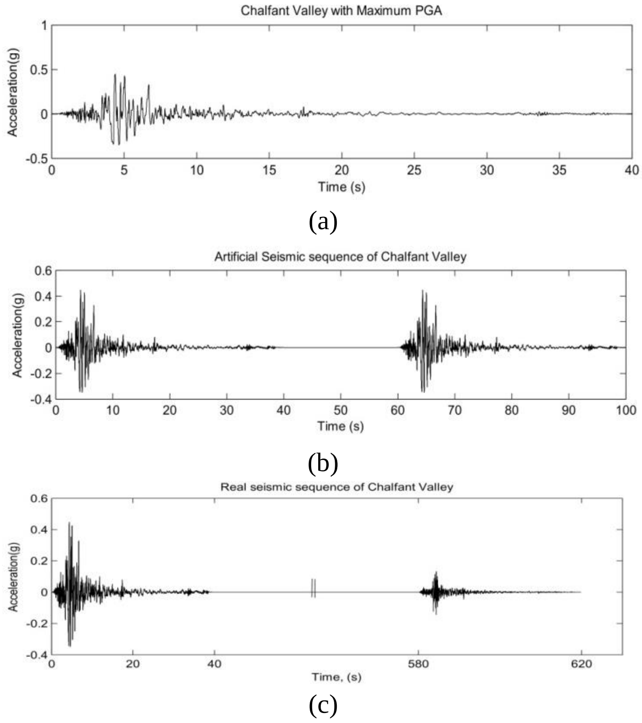


Fig. 1 Chalfant Valley record (a) Only main shock, (b) Artificial sequence, (c) As recorded sequence

2. Research methodology

Today, despite the evidence in Structural and earthquake engineering literature, most structures are designed according to the modern seismic codes which only apply a single earthquake on structure modeling and analysis. In this case, the structure may sustain damage in the event of the “Design earthquake”, and this single seismic design philosophy does not take the effect of strong aftershocks on the accumulated damage of structures into account. However, if this structure is located in seismic regions, not only it will be exposed to a single seismic event, but also to a seismic sequence. The evidence from recent earthquakes confirms this scenario. For example, the Wenchuan earthquake was a deadly earthquake that measured at 8.0 on May12, 2008 in Sichuan province of China. After the main shock, 104 major aftershocks ranging in magnitude 4.0-6.1 were recorded within 72 hours after the main shock. Between May 12 and November 6, there had been 42719 total aftershocks. Among them, 246 aftershocks ranged from 4.0 to 4.9, 34 from 5.0 to 5.9, and 8 from 6.0 to 6.4. Based on this earthquake and other strong ground motions such as Tangshan and Japan with magnitudes 7.8 and 9.0 in 1987 and 2011 respectively, the critical role of aftershocks in structural safety can be confirmed (Huang *et al*. 2014) and the weakness in structural design codes can be shown.

Hence seismic sequence phenomenon has been studied by different methods in recent years. It should be noted that although previous studies developed extensive analyses and provided useful information about the structure’s response under seismic sequences, but the use of artificial seismic sequences could lead to non-conservative prediction of response and behavior of structures under real seismic sequences. For this reason, the present study focuses on the influence of consecutive earthquakes on increased nonlinear damage and response of reinforced concrete frames. In this regard, six concrete moment resisting frames with 3, 5, 7, 10, 12 and 15 stories are designed and analyzed under two different records with seismic sequences from real and artificial cases. Then Park- Ang’s damage index is selected and the result of time history analysis is compared for two cases.

3. Strong ground motions

In this study, attention is focused on increased cumulative damage and response in the RC frames subjected to aftershocks. For this purpose, three groups of strong ground motions with two types of PGA are selected. These records are available in PEER (Pacific Earthquake Engineering Research) center (PEER 2013). First group involves single earthquakes (only main shock) with maximum and approximately maximum PGA, that is in second or third level toward the maximum value. The second contains records with real (as recorded) seismic sequence that it has two subsets: 1) main shock and aftershock with maximum PGA and 2) at least one of them has approximately maximum PGA. In the third, by using main shocks from the first group, records are generated with “Back to back” approach. It’s clear that in this group, the aftershock’s characteristics such as frequency content and time duration are similar to main shocks. It should be noted that all of consecutive records in the second group - both of main shock and aftershock in each sequence - not only occurred in similar directions and same stations, but also their real time gap is less than 10 minutes. For example,

Table 1 The features of ground motion records with maximum PGA (PEER 2017)

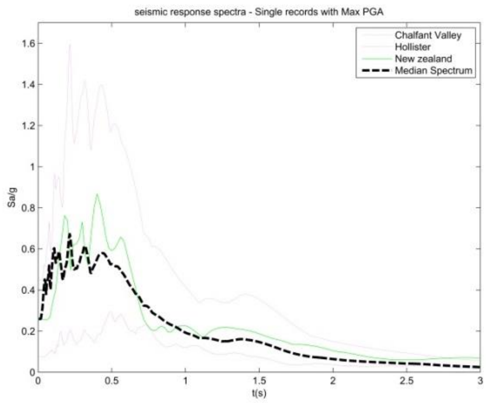
Earthquake	Date	Time	Time gap (min)	Magnitude	Station	PGA
Chalfant Valley	86-07-21	14:42	9	6.19	CDMG 54428 Zack Brothers Ranch	0.4246
	86-07-21	14:51		5.65	CDMG 54428 Zack Brothers Ranch	0.1347
Hollister	61-04-09	7:23	2	5.6	USGS 1028 Hollister City Hall	0.121
	61-04-09	7:25		5.5	USGS 1028 Hollister City Hall	0.0683
New Zealand	87-03-02	1:42	9	6.6	99999 Matahina Dam	0.2926
	87-03-02	1:51		5.8	99999 Matahina Dam	0.0525

Table 2 The features of ground motion records with approximately maximum PGA (PEER 2017)

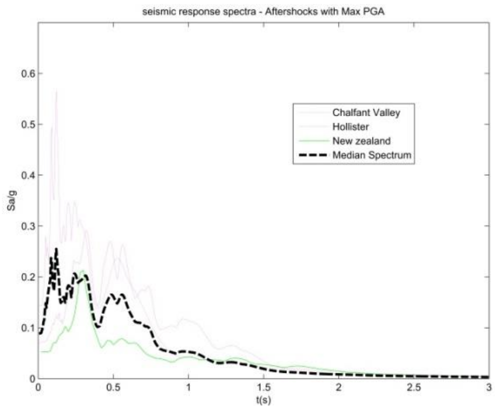
Earthquake	Date	Time	Time gap (min)	Magnitude	Station	PGA
Chalfant Valley	86-07-21	14:42	9	6.19	CDMG 54171 Bishop - LADWP South St	0.2058
	86-07-21	14:51		5.65	CDMG 54171 Bishop - LADWP South St	0.0864
Chi- Chi, Taiwan	99-09-20	17:57	6	5.9	CWB 99999 TCU079	0.3308
	99-09-20	18:03		6.2	CWB 99999 TCU079	0.3022
Imperial Valley	79-10-15	23:16	3	6.53	USGS 952 El Centro Array #5	0.4481
	79-10-15	23:19		5.01	USGS 952 El Centro Array #5	0.2374
Irpinia, Italy	80-11-23	19:34	1	6.9	ENEL 99999 Sturno	0.2898
	80-11-23	19:35		6.2	ENEL 99999 Sturno	0.076
Northridge 1	94-01-17	12:31	1	6.69	CDMG 24279 Newhall - Fire Sta	0.698
	94-01-17	12:32		6.05	CDMG 24279 Newhall - Fire Sta	0.0407
	94-01-17	12:41	9	5.2	CDMG 24279 Newhall - Fire Sta	0.1632
Northridge 2	94-01-17	12:31	1	6.69	CDMG 24278 Castaic - Old Ridge Route	0.4898
	94-01-17	12:32		6.05	CDMG 24278 Castaic - Old Ridge Route	0.0231
	94-01-17	12:41	9	5.2	CDMG 24278 Castaic - Old Ridge Route	0.0389

Fig. 1 shows Chalfant Valley record in three cases: only main shock, as recorded sequence and artificial sequence (back to back method). The ground motion features are presented in Tables 1-2. Moreover, the seismic response spectra and the corresponding mean spectrum of main shocks and aftershocks for analyzing the frames are shown in Fig. 2.

A sample of the nonlinear behavior of the 5 story frame in the left span under Northridge earthquake (1994) is illustrated in Fig. 3. These curves are the Moment- Rotation of the left hinges in first and third story. As shown in this figure, according to the backbone curve, the concrete frame goes through from the linear region and makes loops.



(a)



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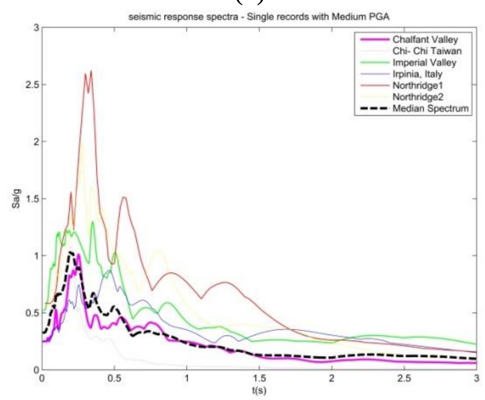
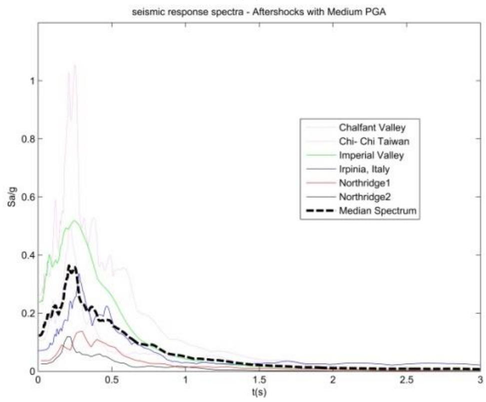


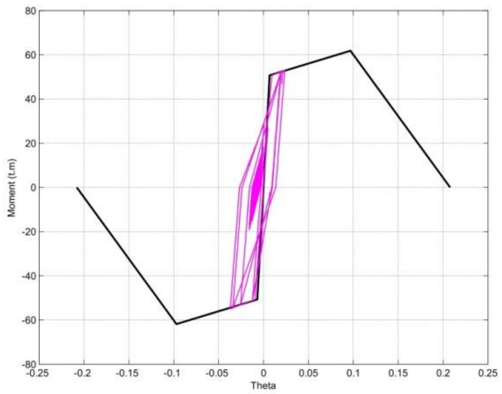
(c)



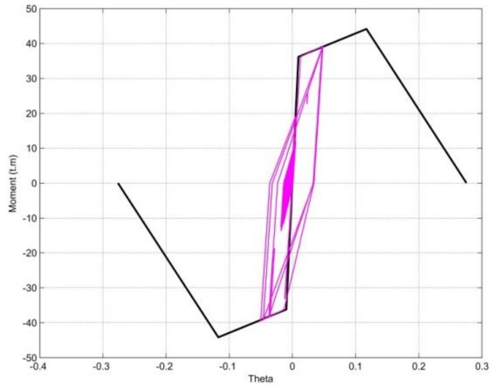
(d)

Fig. 2 The seismic response spectra and the corresponding mean spectrum, (a) and (b) Main shocks and aftershocks with Max PGA, (c) and (d) Main shocks and aftershocks with approximately maximum PGA

4. Modeling



(a)



(b)

Fig. 3 Moment-rotation curve for 5 story frame in left span, (a) First story, (b) Third story

A set of 2-D concrete moment resisting frames with 3, 5, 7, 10, 12 and 15 stories and Intermediate ductility is designed under vertically regular condition (stiffness and strength) in Tehran, capital of Iran - with 375 (m/s)≤*Vs*≤750 (m/s) - according to these steps:

- Determination of Gravity loads based on Standard No. 2800 static linear guidelines.
- Linear static analysis with ETABS software by introduction of seismic coefficient of earthquake (Coefficient *C* in calculating of the base shear force: *V*=*CW*) and determination of the most appropriate concrete sections of beams and columns.
- Modeling of these sections with the most appropriate behavior models and compatible materials with these models in Opensees software.
- Nonlinear dynamic analysis with three seismic scenarios (Single record, earthquakes with real and artificial seismic sequence) by OpenSees (Opensees 2007) software which is an Open System for Earthquake Engineering Simulation and contains a comprehensive archive of linear and nonlinear behavior in material and also concrete or steel elements definition. In addition, this software can introduce different elements for modeling of structural and non- structural sections. In this regard, for compatibility aspects between the seismic analysis and seismic design, selected records were scaled by designing spectrum for each structure fundamental period in order to have identical spectral acceleration. For this purpose, linear scaling (Atkinson 2009) is used to scale all ground motion records by multiplying time histories by the appropriate factor (Hancock *et al*. 2008). This approach is easy to understand and perform. Also, the frequency content and original phasing of the records are preserved in this method (Atkinson 2009).
- Processing the output results of nonlinear dynamic analysis with MATLAB software

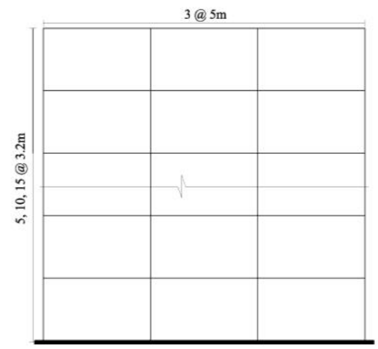


Fig. 4 The schematic elevation of the studied frames

The story height and the bay lengths are 500 and 320 cm, respectively (Fig. 4). The gravity loads were selected based on the values typically employed in engineering practices. Therefore, 3, 5, 7, 10, 12 and 15 story frames have masses equal to 228, 399, 586, 693, 855 and 1102 tons and natural periods equal to 0.86, 1.04, 1.36, 2, 2.4 and 3.17 (s), respectively. Further, equivalent lateral design forces were determined based on Standard No. 2800 guideline i.e. the design spectrum for each site class was derived, and the corresponding design base shears were calculated. Both gravity and seismic loads were then imposed on the frames according to the counteractive load combinations of ASCE7-05 standard (the reader is referred to Section 12.4.2.3 of ASCE7-05 for further details: (0.9-0.2)*D*+1.0Q*E*). The geometric and material properties of the designed 3, 5, 7, 10, 12 and 15 story frames are presented in Tables 3-4. The seismic nonlinear behavior of frame elements plays the most important role in the global behavior of the considered frames. As previously mentioned, the Open Sees framework has been used for response history analysis. In the present paper, the nonlinear beam element with concentrated hinges is employed for the beam modeling. Beams with concentrated plastic hinges and columns with fiber section are employed to simulate the nonlinear flexural behavior of the moment frames. The beam with hinges element is chosen for the beams. Thus, a predetermined length at both ends was allocated to the plastic hinges, and an elastic material was assigned to the mid span.

As the nonlinear behavior was assumed to be focused in the hinges, expansion of the non-linearity to the elastic region was less likely to happen. Therefore, the coefficient of cracking was set to be 0.5 for the elastic segment of the beams. The nonlinear behavior of the plastic hinges was defined in accordance with Haselton *et al.* (2007), who proposed essential relationships in their study based on the calibration of numerous test results in the form of the tri-

Table 3 Geometric properties of the designed 3, 5, 7, 10, 12 and 15 story frames

Number of story	Level	Column Width (cm)	Column Height (cm)	Beam Width (cm)	Beam Height (cm)
3	1, 2, 3	40	40	40	35
5	1, 2	50	50	50	40
	3, 4, 5	40	40	50	40
7	1, 2, 3, 4	55	55	55	45
	5, 6, 7	45	45	45	35
10	1, 2, 3, 4	55	55	55	40
	5, 6, 7	45	45	45	40
	8, 9, 10	40	40	40	35
12	1, 2, 3, 4	60	60	60	50
	5, 6, 7, 8	55	55	55	40
	9, 10, 11, 12	40	40	40	35
15	1, 2, 3, 4	65	65	65	50
	5, 6, 7, 8	55	55	55	50
	9, 10, 11, 12	45	45	45	40
	13, 14, 15	35	35	35	35

Table 4 Material properties of the designed 3, 5, 7, 10, 12 and 15 story frames

Specified Concrete Compression Strength, f'_c (kg/cm^2)	Modulus of Elasticity, E (kg/cm^2)	Yield Stress, F_y (kg/cm^2)
250	2.388 e+5	4000

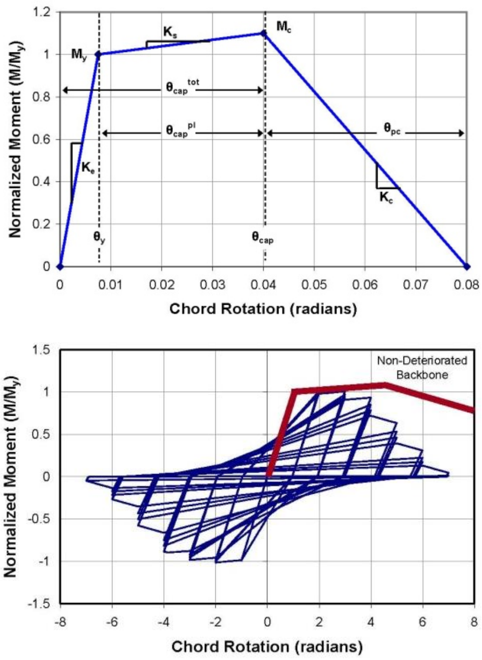


Fig. 5 Monotonic behavior and tri-linear backbone curve suggested by Ibarra (Haselton *et al*. 2007)

linear backbone curve suggested by Ibarra (Fig. 5).

An important feature of the model is that softening due to concrete crushing, reinforcement buckling and yielding and bond slip can be considered in the negative stiffness region, namely the post cap behavior. The tri-linear Ibarra

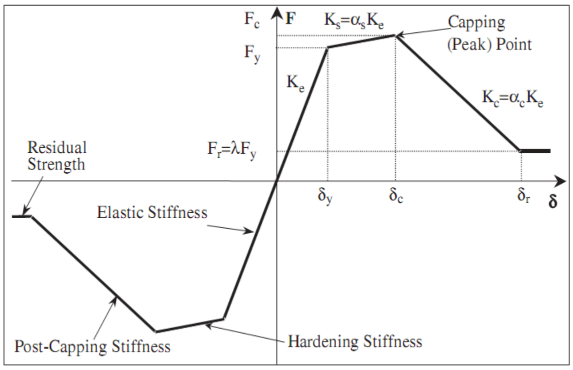


Fig. 6 Backbone curve of Clough material (Haselton *et al*. 2007)

model, as mentioned above, was employed in the Open Sees platform using the Clough material (Fig. 6) proposed by Altoontash (Haselton *et al*. 2007). Then uniaxial sections with pre-defined M-θ according to the Clough material were assigned to the plastic hinges. It should be noted that all parameters calculated to form the Ibarra model were in terms of rotations. Thus, in order to make them applicable to a beam With Hinge element, the simple equation $\varphi=\theta/L$ (φ curvature, θ rotation and L plastic hinge length) was used to transform rotations into curvatures (Scott and Fenves 2006). This is an advantage of the selected beam element. The plastic hinge's length was set to equal the beam's height for all the cases.

Columns were modeled by means of the fiber method with the capability of developing distributed plasticity along the element's length. This choice was made mostly due to the fact that the flexural behavior in the columns is highly dependent on the interaction of their axial and bending forces. However, the aforementioned approach for beams was incapable of considering variable axial forces during the analysis. As a result, the fiber sections were assigned to the nonlinear Beam Column elements. Each element was also divided into four sub-elements in a story level to provide more robustness. In the following, reinforced concrete frames are verified by the analytical and experimental results obtained from parametric study (Huang *et al*. 2014) and a test of structure on shaking table (Nagae *et al*. 2015). In Nagae study, a full scale reinforced concrete building - four stories - has been tested on the E-Defense shake table provided by E-Defense Company - National Research Institute for Earth Science and Disaster Prevention (NIED) -in Japan.

It should be noted, classical damping which is an appropriate idealization in multistory building with a similar structural system and structural materials over its height, is considered by Rayleigh damping in all nonlinear dynamic analysis of this paper.

The stiffness-proportional term of Rayleigh damping was examined by Charney (2006) and three alternatives proposed to define damping matrix [C]: (1) based on the initial elastic stiffness matrix and held constant during the analysis; (2) based on the tangent (softened) inelastic stiffness matrix and updated throughout the analysis using the initial proportionality constants, a_M and a_K; and (3) based on the tangent stiffness matrix and updated throughout the analysis, where the proportionality constants, *aM* and *aK* are updated to maintain a specified critical damping percentage for the inelastic vibration modes. A number of researchers, such as Charney (2006) and Petrini *et al*. (2008), have advocated the second option (i.e., fixing proportionality constants applied to the updated tangent stiffness matrix) as a practical way to avoid excessive damping in inelastic analysis, but others have countered that the first option (i.e., constant damping) is legitimate. In Rayleigh procedure a_0 and a_1 (Eq. (1)) are mass-proportional damping and stiffness-proportional damping constants.

$$C = a_0 M + a_1 K \tag{1}$$

In any one mode, these constants are calculated based on Eq. (2)

$$a_0 = 2\zeta_i \omega_i \quad , \quad a_1 = \frac{2\zeta_i}{\omega_i} \tag{2}$$

Where, ζ_i is damping ratio in mode "*i*". For example, the coefficients a_0 and a_1 for the first mode of the three-story RC frame are 0.73 (1/s) and 0.0137 (s), respectively.

For two modes *i* and *j*, these constants are calculated based on Eq. (3)

$$\begin{gathered}\frac{1}{2}\begin{bmatrix}1/\omega_i & \omega_i\\ 1/\omega_j & \omega_j\end{bmatrix}\begin{Bmatrix}a_0\\ a_1\end{Bmatrix}=\begin{Bmatrix}\zeta_i\\ \zeta_j\end{Bmatrix}\xrightarrow{\text{if } \zeta_i=\zeta_j}\\ a_0=\zeta\frac{2\omega_i\omega_j}{\omega_i+\omega_j}\ ,\ a_1=\zeta\frac{2}{\omega_i+\omega_j}\end{gathered} \tag{3}$$

Therefore, the coefficients a_0 and a_1 for the first two mode of the three-story RC frame are 0.56 (1/s) and 0.0032 (s), respectively. Also, in order to consider the stiffness deterioration in reinforced concrete frames during an earthquake, Ibarra model with tri-linear backbone curve has been used. Hence, fixing proportionality constants applied to the updated tangent stiffness matrix can be used in the damping matrix determination. For example, in the beginning of earthquake which the stiffness deterioration has not been still happened, the damping matrix [*C*] for the three-story RC frame is shown in Eq. (4)

$$\begin{gathered}M=\begin{bmatrix}81.74 & 0 & 0\\ 0 & 81.74 & 0\\ 0 & 0 & 64.78\end{bmatrix},\ K=7462.5\begin{bmatrix}2 & -1 & 0\\ -1 & 2 & -1\\ 0 & -1 & 1\end{bmatrix}\\ a_0=0.56(1/s),\ a_1=0.0032(s)\\ C=a_0 M + a_1 K \rightarrow C=\begin{bmatrix}93.534 & -23.88 & 0\\ -23.88 & 93.534 & -23.88\\ 0 & -23.88 & 60.157\end{bmatrix}\end{gathered} \tag{4}$$

5. Damage index

Park and Ang's global damage index (Park *et al*. 1985) is one of the best known and most widely used damage indexes. It is based on scaled values of ductility and dissipated energy of the local element during the seismic ground shaking. The damage index (DI) is defined as a combination of maximum deformation and hysteresis energy

$$DI = \frac{\delta_m}{\delta_u} + \frac{\beta}{P_y \delta_u}\int dE_h \tag{5}$$

Where δ_m is the maximum deformation of the element during the earthquakes, δ_u, the ultimate deformation, β is a model constant parameter (usually, β=0.05−0.20) to control strength deterioration, $\int dE_h$, the hysteresis energy absorbed by the element during the earthquake and P_y is the yield strength of the element. In order to calculate the Damage index -based on Eq. (5)- β is taken as 0.15 according to Park *et al*. (1987) for nominal strength deterioration. Damage degree classification of Park Ang's damage index is shown in Table 5. In 1997, Park *et al*. have proposed a correlation between local signs of damage and five levels of damage, according to Table 6. According to these tables, DI>1 means partial or total collapse.

Table 5 Damage degree classification of Park Ang's damage index (Park and Ang 1987)

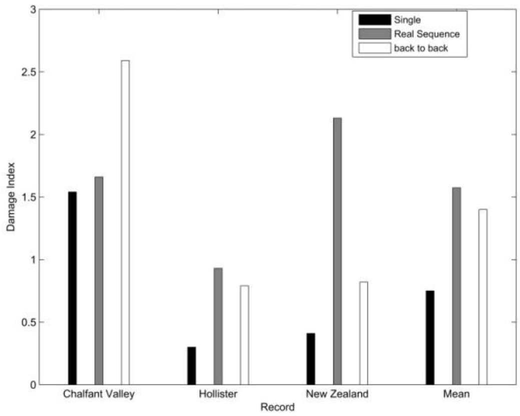
Damage Degree	Damage's physical description
Light	Minor, localized, fissures/cracks.
Minor	Minor fissures/ cracks localized throughout the entire structure. Local crushing of concrete.
Moderate	Cracks on large surfaces. Failure of flexible reinforced concrete elements.
Sever	Failure of flexible reinforced concrete elements throughout the entire structures. Column's reinforcement buckling.
Total	Partial or total collapse.

Table 6 Normalized damage index ranges for a five-level scale (Kunnath 1997)

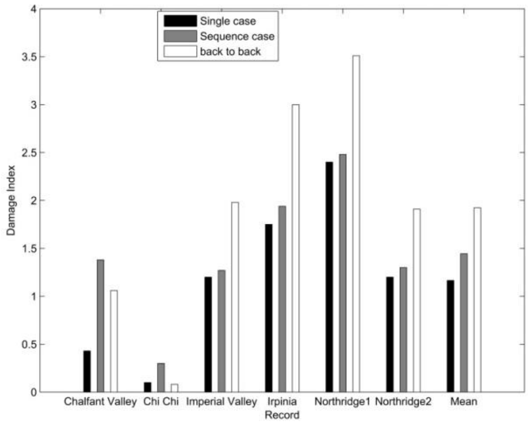
Damage level	No damage	Light damage	Moderate damage	Strong damage	Collapse
Range of DI	0-0.1	0.1-0.24	0.25-0.4	0.4-1	>1

6. Results

After selection of records and Park- Ang's damage index, first the gravity loads are applied and then the ground motions are used in order to perform the nonlinear analysis of frames. In the static analysis under gravity loads, Newton's solution algorithm was used. This algorithm updates tangent stiffness at every iteration. In this regard, first load increment (pseudo-time step), number of load step, maximum number of iterations and tolerance are 0.1, 10, 10 and 1.0e-8, respectively. It should be noted, various types of Newton-Raphson method were used in nonlinear dynamic analysis under seismic scenarios. Norm Displacement Increment Test is selected as convergence test with tolerance 1e-3 and maximum number of iterations 100 that will be performed before "failure to converge" is returned. In this paper, results are presented as damage index, maximum rotation in the beam hinge and maximum story drift. Furthermore, in order to carry out a comprehensive study in the next step, the average of the results is examined.

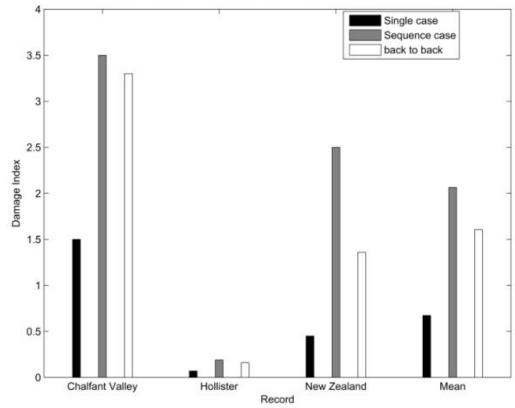


(a) Scenarios with maximum PGA

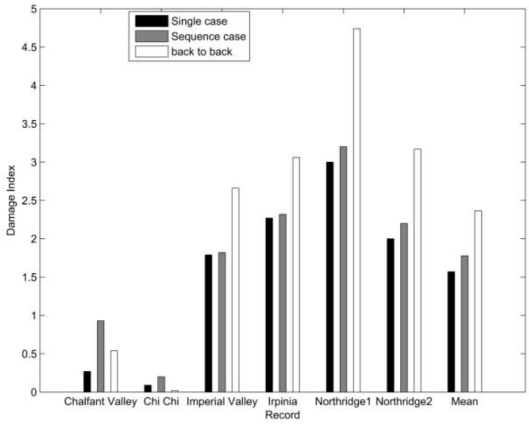


(b) Scenarios with approximately maximum PGA

Fig. 7 Damage index of 5 story frame under single earthquake, as recorded and artificial sequences



(a) Scenarios with maximum PGA



(b) Scenarios with approximately maximum PGA

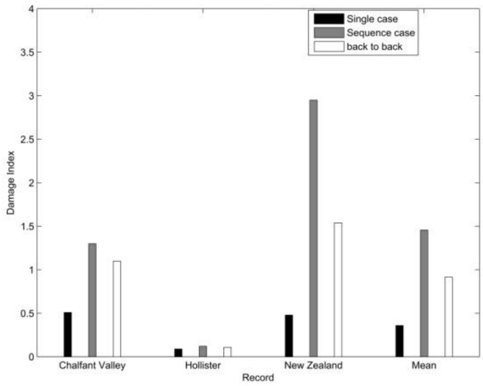
Fig. 8 Damage index of 10 story frame under single earthquake, as recorded and artificial sequences

6.1 Structural damage

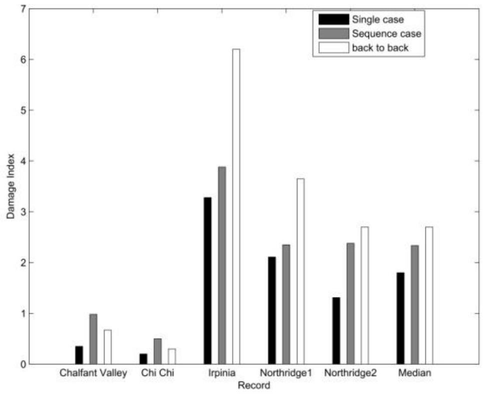
Damage index is calculated for all reinforced concrete frames under three seismic scenarios with maximum and approximately maximum PGA. Based on Eq. (5), the hysteresis energy absorbed during the earthquake (E) is one of the most effective parameters on Park Ang's damage index. Due to the accumulated damage caused by previous earthquake, structural strength of RC frames was decreased. Therefore, RC frames under successive earthquake have been formed wider hysteresis loops rather than single scenarios. It is clear that the area under RC frames force-displacement curves under successive earthquake is more than RC frames's under single earthquake which results in higher Park Ang's damage index. According to Tables 5-6, DI>1 means partial or total collapse. In the present paper, RC frames were subjected to different repeating main shock-aftershock earthquake sequences. In most cases, structural damage caused by single and successive scenarios was leaded to partial collapse in RC frames. For example, 15 stories frame was completely collapsed under artificial and real successive scenario of Imperial Valley (1979) earthquake. The results of analysis for 5, 10 and 15 story frames are compared in Figs. 7- 8 and 9. In these figures, bars denote the damage index of frames under only main shock, real and back to back sequence. The last three bars show the average of results for mentioned scenarios. It's clear that the seismic sequence phenomenon - real and artificial method - increases the damage state of structures when compared to single ground motion because of cumulative damage before any repair is possible. Also, the use of artificial methods as back to back approach for seismic sequence generation can lead to non-conservative prediction of response structures. Therefore, the performance of structures will be disrupted.

6.2 Maximum structure drift

In this step, maximum drift of frames under three scenarios are compared in order to examine the importance of the seismic sequence phenomenon. For a more comprehensive understanding of results, this parameter is compared in artificial sequence to real. For example, results are shown for 5 and 15 story frames in Figs. 10-11. The ratio of maximum drift (back to back/real) is larger than one. It shows that the using of seismic sequence in structural design is very important, while unfortunately current seismic codes do not account for its effects. In addition, to achieve an appropriate estimation of structure's behavior under seismic scenarios, it's better to use real sequences. So that, unsuitable estimation might occur if the relationships of the ground motion characteristics between the main shock and the following aftershocks are not properly represented in the artificial sequences.



(a) Scenarios with maximum PGA



(b) Scenarios with approximately maximum PGA

Fig. 9 Damage index of 15 story frame under single earthquake, as recorded and artificial sequences

6.3 Maximum rotation in beam hinges

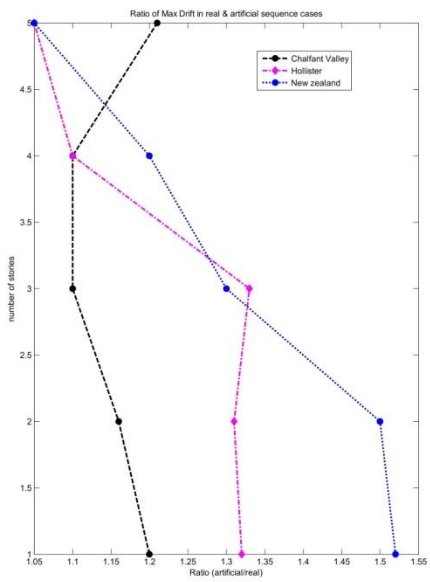
Maximum rotation in beam hinges is another parameter that is used in this study. It is calculated in each story of frames under strong ground motions with artificial and real aftershocks. Then, the ratio of maximum rotation in artificial case to real is compared for records with maximum and approximately maximum PGA. A sample of this result for 5 story frame is illustrated in Fig. 12. In this figure, vertical and horizontal axes are "number of stories" and the ratio of maximum rotation respectively. As shown, the ratio (back to back/real) is larger than one. It's another confirmation for the necessity of considering the seismic sequence in structural design.

6.4 Comparison between average results

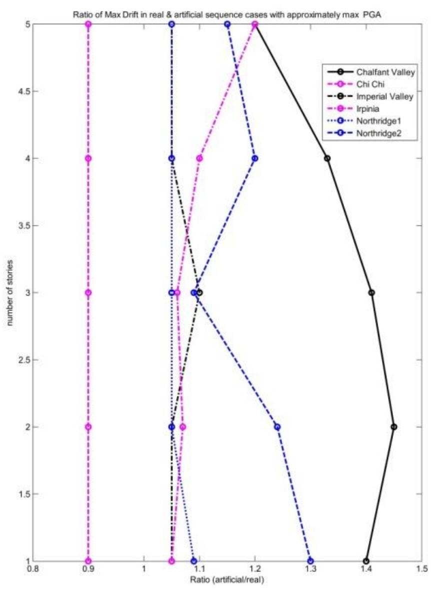
This paper presents the results of an analytical investigation aimed at evaluating the cumulative damage of six RC frames subjected to different repeating main shock-aftershock earthquake sequences. So, in the last step, average of results for all frames under three groups of seismic scenarios with maximum and approximately maximum PGA is compared.

6.4.1 Mean damage index

The average of Park- Ang's damage index is calculated for 3, 5, 7, 10, 12 and 15 story frames under three groups of scenarios with maximum and approximately maximum PGA (Fig. 13). As shown, in both cases - maximum and approximately maximum PGA - the seismic damage for records with sequence is higher than that for single ground motions. But in case (a) damage caused by real strong ground motions is more than artificial earthquakes. While in case (b) it is less. Fig. 14 shows the average ratio of Park Ang's damage index for all reinforced concrete frames under seismic sequence to single earthquake. It's clear that the frames under consecutive records with maximum PGA - both of real and artificial cases - are damaged much more than single records and this parameter increases with increasing the height of the structure. Such that, Park- Ang's damage index for "3, 5, 7" and "10, 12" and 15 story frames

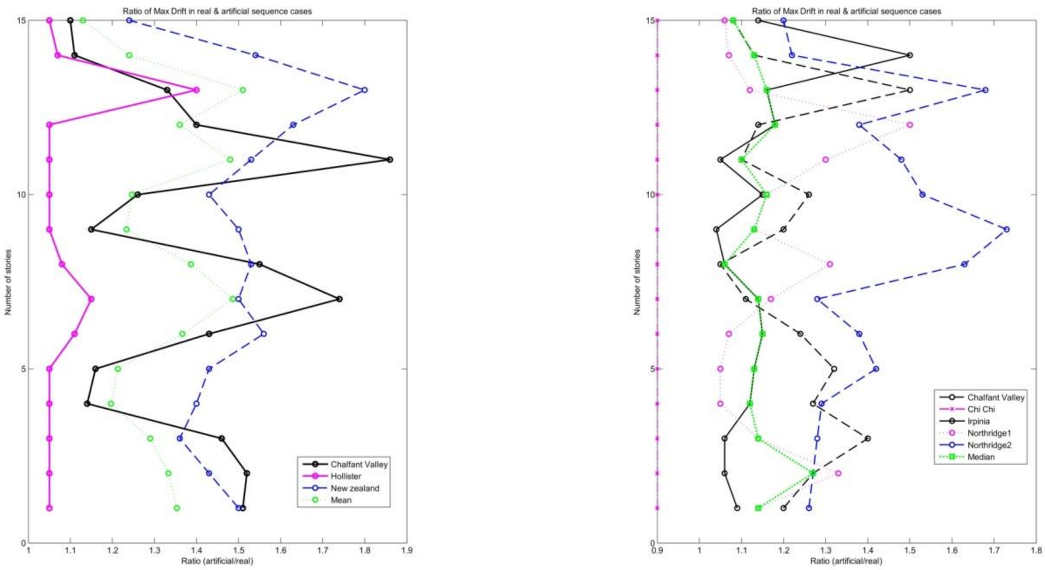


(a) Scenarios with maximum PGA



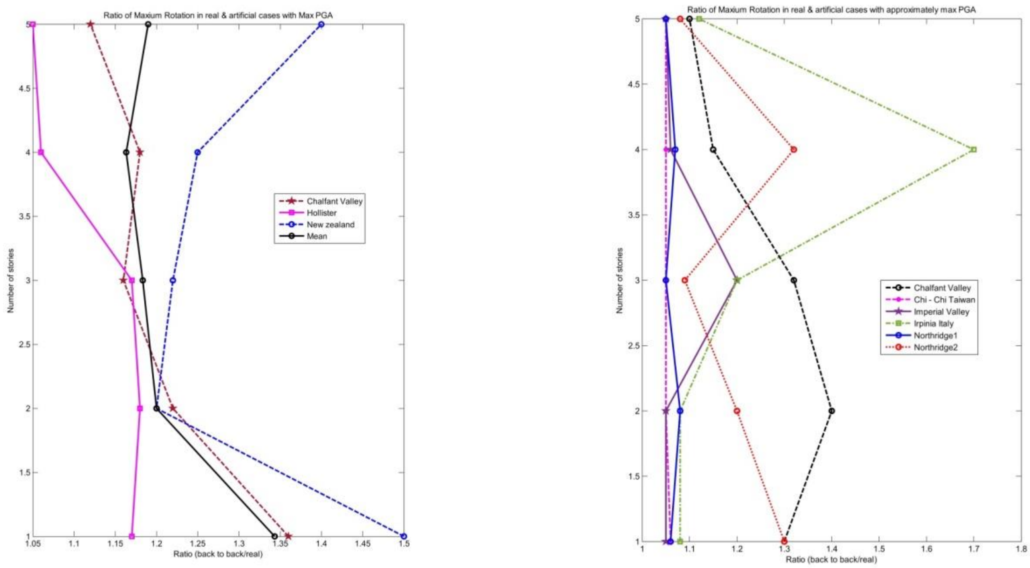
(b) Scenarios with approximately maximum PGA

Fig. 10 Ratio of maximum drift for 5 story frame under artificial and real sequences



(a) Scenarios with maximum PGA (b) Scenarios with approximately maximum PGA

Fig. 11 Ratio of maximum drift for 15 story frame under artificial and real sequences



(a) Scenarios with maximum PGA (b) Scenarios with approximately maximum PGA

Fig. 12 Ratio of maximum rotation for 5 story frame under artificial and real sequences

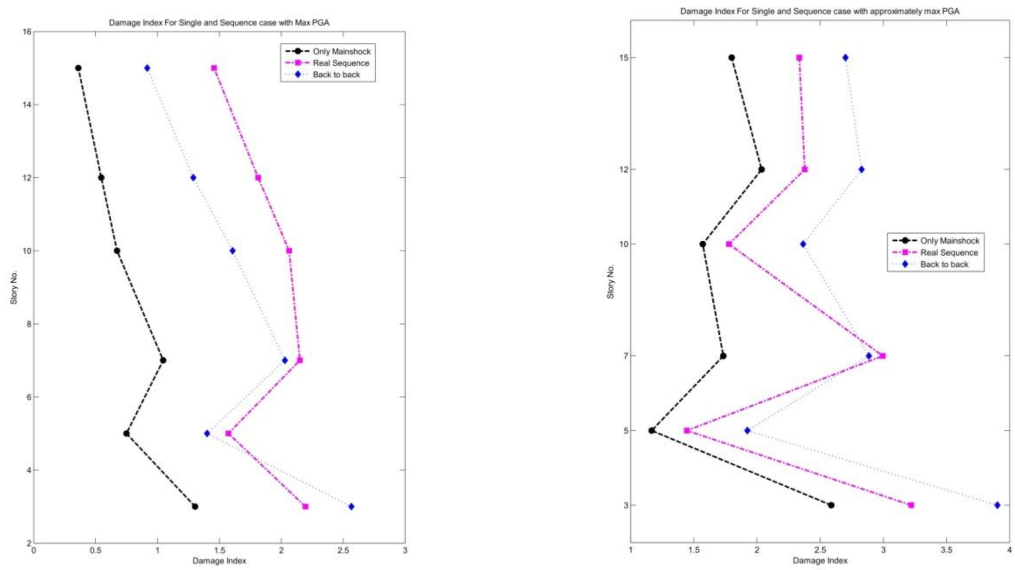
subjected to real aftershocks with the maximum PGA is 2, 3 and 4 times larger than that for single earthquakes, respectively. While, the damage of frames has increased about 2.2 times when artificial aftershocks with maximum PGA are considered. On the other hand, taller structures are more vulnerable to seismic sequence phenomena, especially in real cases. For artificial consecutive records with approximately maximum PGA, this ratio is higher than that for real case. In summary, the seismic damage of real and artificial main shock- aftershock earthquake is higher than that for single ground motion. The ratios that are larger than one, confirm this matter. For example, the damage in frames is 1.2 and 1.5 times larger when real and artificial consecutive earthquakes with approximately maximum PGA are induced, respectively.

6.4.2 Mean maximum ratio of story drift

Fig. 15 shows the mean maximum ratio of story drift for all frames under artificial seismic sequence to single records. According to this figure, because of aftershocks with maximum PGA, lateral displacement in short (3 and 5 story frames), medium (7 and 10 story frames) and tall structures (12 and 15 story frames) are critical in lower, middle and upper stories respectively. However, this matter is true with less sensitivity for strong ground motions with approximately maximum PGA and in all cases; the ratio is larger than one.

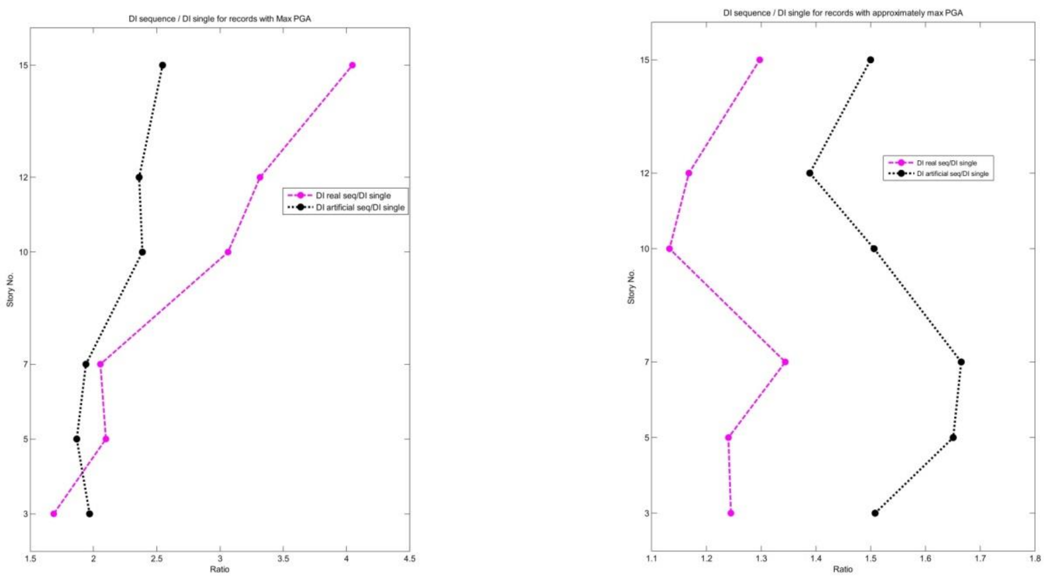
6.4.3 Mean maximum ratio of beam rotation

Fig. 16 shows the ratio of mean maximum rotation in 3,5, 7, 10, 12 and 15 story frames under artificial seismic



(a) Records with maximum PGA (b) Records with approximately maximum PGA

Fig. 13 Comparison between the mean damage index of frames under seismic scenarios



(a) Records with maximum PGA (b) Records with approximately maximum PGA

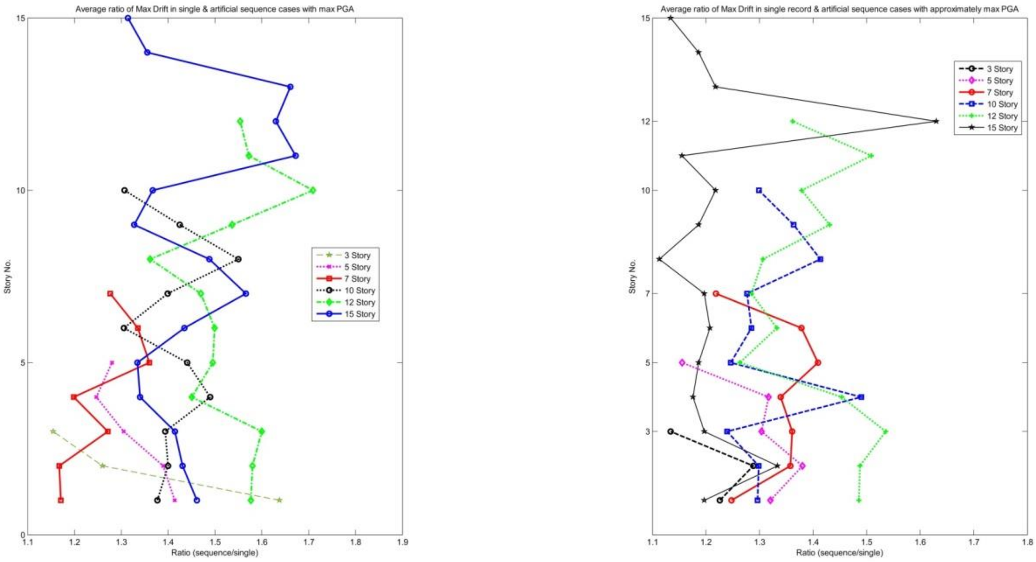
Fig. 14 Comparison between the average ratio of damage index for frames under seismic scenarios

sequence to real consecutive records. As shown in this figure, because of aftershocks with maximum PGA, rotation in short and tall structures are critical in lower and upper stories respectively. However, the set of reinforced concrete frames is sensitive in upper stories under consecutive earthquakes with approximately maximum PGA.

7. Conclusions

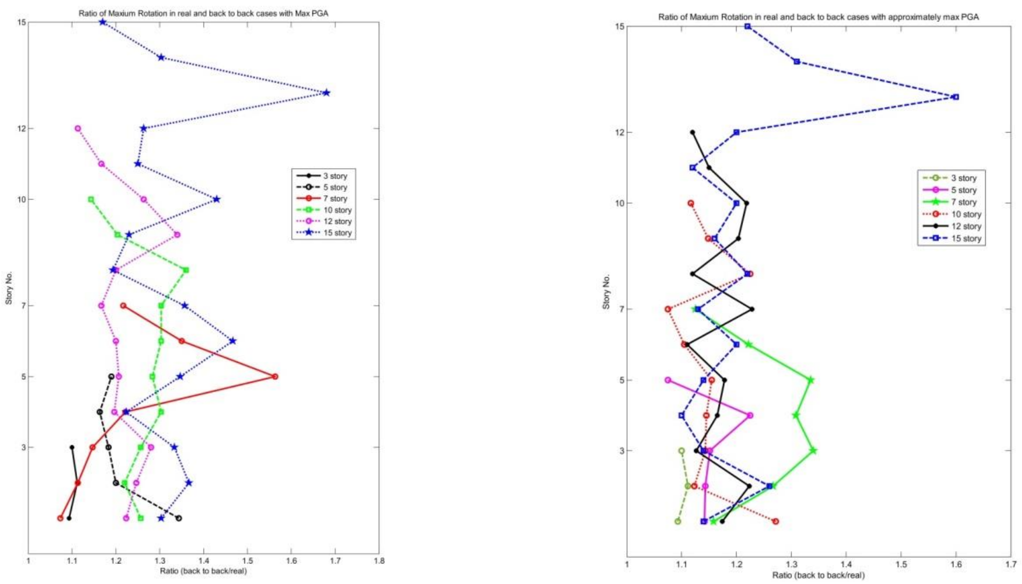
This paper presents the results of analytical investigation aimed at evaluating the increased damage and response of 3, 5, 7, 10, 12 and 15 story RC frames subjected to real and artificial consecutive earthquakes. For this, some parameters such as Park- Ang's damage index, maximum drift and maximum rotation are selected. From the results obtained in this investigation, the conclusions are drawn as followings:

- The seismic damage of real and artificial main shock-aftershock earthquake is higher than that for single ground motion. The ratios that are larger than one, confirm this matter. For example, the damage in frames is 1.2 and 1.5 times larger when real and artificial consecutive earthquakes with approximately maximum PGA are induced, respectively.
- In the structures under consecutive earthquakes with maximum PGA, the seismic damage increases with increasing the height of the structure. Such that, Park-Ang's damage index for "3, 5, 7" and "10, 12" and 15 story frames subjected to real aftershocks with the



(a) Records with maximum PGA (b) Records with approximately maximum PGA

Fig. 15 Comparison between the ratio of maximum story drift’s mean for frames under seismic scenarios



(a) Records with maximum PGA (b) Records with approximately maximum PGA

Fig. 16 Comparison between the ratio of maximum rotation’s mean for frames under seismic scenarios

maximum PGA is 2, 3 and 4 times larger than that for single earthquakes, respectively. While, the damage of frames has increased about 2.2 times when artificial aftershocks with maximum PGA are considered. As a result, taller structures are more vulnerable to seismic sequence phenomena, especially in real cases.

• The ratio of maximum lateral displacement in short - 3, 5 and 7 story frames- and medium - 7 and 10 story frames- and tall structures -12 and 15 story frames- under consecutive earthquakes with maximum PGA is critical in lower, middle and upper stories to single case respectively. However, this matter is true with less sensitivity for strong ground motions with approximately maximum PGA and in all cases; the ratio is larger than one.

• Tall reinforced concrete frames are sensitive in upper stories under consecutive earthquakes with maximum and approximately maximum PGA.

• The damage state of the structures may strongly depend on the cumulated damage caused by previous earthquake due to accumulation of inelastic deformations before any repair is possible. The main shock- aftershock seismic phenomena should be taken into account to estimate the performance of the structure reliability. Since one aftershock -even with medium intensity- can have an effect on structural damages.

• The use of artificial approaches such as “Back to back method” in order to introduce consecutive records as design earthquake can lead to non-conservative prediction of performance and damage of structures under real seismic sequences. This situation might occur if the relationships of the ground motion characteristics between the main shock and the following aftershocks are not properly represented in the artificial sequences. For example, maximum rotation in beam hinges under artificial sequences is much more than that for real cases. Furthermore, the seismic damage caused by artificial consecutive strong ground motions with maximum and approximately maximum PGA is less and more than that for real records respectively. All of the abovementioned confirms the inefficiency of this method.

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