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Performance Evaluation of the Damaged Steel Moment Frames under Mainshock-Aftershock Sequences Considering Plastic Hinge Modification Factors

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ABSTRACT

With the aim of providing a tool for comprehensive seismic analyses of damaged steel moment resisting frames, this paper presents the development of a structural model that is able to describe the general behavior of damaged steel structures under repeated earthquakes. A regular 3D special steel moment frame designed using AISC 360-10 have been modeled to evaluate the effects of mainshock-aftershock sequences on seismic behavior of structures. At first, rotations of each plastic hinge of the structure were determined under a set of records by the nonlinear dynamic analysis. Next, based on the deformation of hinges under main-shocks, suitable damaged hinges were assigned to each hinge. At last, again, the nonlinear time history analyses were carried out to evaluate the performance of damaged steel moment frame structures under repeated earthquakes. It seems that the effects of cumulative damage in Damaged Modified Method did not trigger special floors. The results have shown the importance of considering the effects of mainshock-aftershock sequences in design codes. Finally, the response of the structure computed by Damaged Modified Hinges method was compared with repeated method that is common in evaluation and vulnerability of structures under seismic sequences. Besides, it is shown that considering the initial damage in element behaviors could change the structural mechanism.

Keywords:
Damaged Modified Hinges Method; Repeated method; Structural mechanism; Mainshock-aftershock sequences

1. Introduction

There are several investigations aimed at studying the effects of seismic sequences on the response of structures [1-11]. The first type of those researchers has been focused on the nonlinear responses of single-degree-of-freedom (SDOF) systems such as Mahin [1], Sunasaka and Kiremidjian [2], Amadio et al. [3], Luco et al. [4], Hatzigeorgiou, etc. [5-6].

The other one studied the response of multiple-degree-of-freedom (MDOF) systems such as Fragiacomo et al. [7], Lee and Foutch [8], Li and Ellingwood [9], Ruiz-Garcia et al. [10], Hatzigeorgiou, etc. [11].

Li and Ellingwood [9] studied the response of two steel moment frames having 9 and 20-story steel moment frames that were designed as a typical office building in Los Angeles area prior to the 1994 Northridge earthquake. This article proposed a special way to model mainshocks-aftershocks sequences. Ruiz-Garcia et al. [10] investigated the seismic behavior of some RC highway bridges under as-recorded mainshock-aftershocks sequences. Hatzigeorgiou and Liolios [11] evaluated the response of regular and irregular RC frames under as-recorded and artificial seismic sequences. It had shown that the displacement ductility demands of structures were enhanced under the real seismic sequences.

In all of the referenced papers above, there is a point of ambiguity; i.e. the moment-rotation relations of the deformation controls elements during the mainshock-aftershock sequences are constant. It seems that this obscure procedure or inaccuracy has been cited as a criticism of back-to-back analyses of structures subjected to mainshock-aftershock sequences. The major coordinate research project described in this paper presents an alternative approach to evaluate the effects of stiffness and strength degradation of special steel moment frames components under mainshock-aftershocks.

2. Plastic Hinge Modification Factors

Several different approaches use to evaluate the seismic behavior of structures. In order to static or dynamic nonlinear analyzes of buildings, with an adoption lumped plasticity model, the behavior of deformation control elements has been properly characterized with a moment rotation relation. The moment rotations of plastic hinges have been idealized with multi-linear curves. The idealizing procedure with a multi-linear curve has been described in some guidelines such as FEMA 356 [12].

In order to analyze damaged steel moment frame buildings, with the assumption of a lumped plasticity model (with a moment rotation relation), the beam and column plastic behavior of damaged steel elements were determined based on [13]. Hosseini Hashemi and Naserpour [13] proposed an analytical approach that is capable of considering the effects of cumulative damage on steel elements subjected to mainshock-aftershock sequences.

As mentioned previously, FEMA plastic hinges are conservative [14]. Consequently, in a relative performance analysis, the degree of conservatism should be identical for both the intact and damaged models to give reliable results to judge the plan of rehabilitation projects. Therefore, the damaged behaviors of structural components have been calibrated to the intact elements that had been calibrated with FEMA proposed behaviors. As a result, the plastic hinges of damaged elements should be modified with the suitable variation of stiffness, strength, and ultimate drifts modification factors. The general shape of moment-rotation relations of damaged and intact elements have been compared and illustrated in Figure (1) [13].

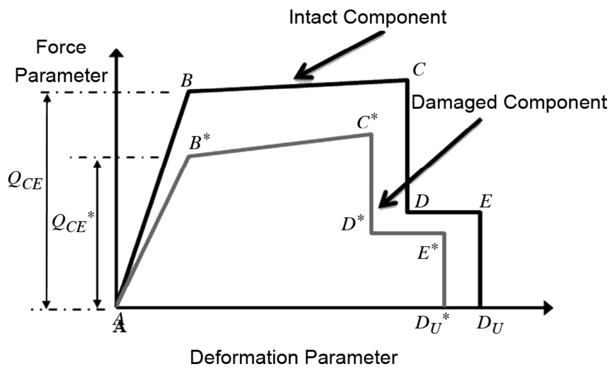

Figure 1. General Shape of Intact and damaged plastic hinges [13].

Modification factors used to modify component properties have been expressed as:

$\lambda_k =$ the modification factor for the effective initial stiffness $(K' = \lambda_K \times K)$.

$\lambda_Q =$ the modification factor for the strength $(Q'_{CE} = \lambda_Q \times Q_{CE})$.

$\lambda_D(a) =$ the modification factor to consider the plastic rotation in point 'a' $(D' = \lambda_D \times D)$.

$\lambda_D(b) =$ the modification factor to consider the plastic rotation in point 'b' $(D'_{ultimate} = \lambda_D \times D_{ultimate})$.

Point 'a' and 'b' have been shown in Figure (2). It should be noted that the notation λ is used to

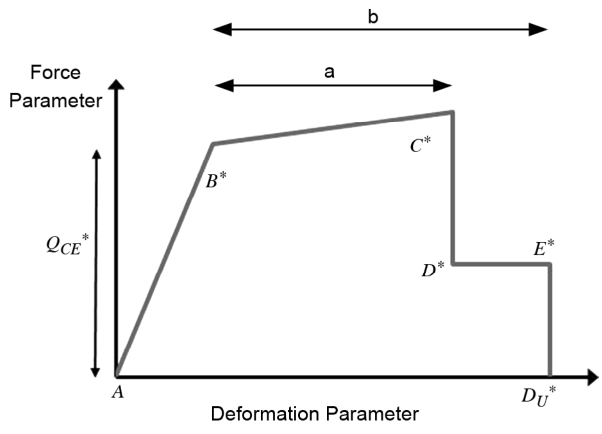

Figure 2. Position of damaged plastic hinges points on moment-rotation relation [13].

alternate intact components properties to damaged component properties [13].

The strength modification factors calculated for the box column subjected to 1% to 4% initial drift are shown in Figure (3) [13].

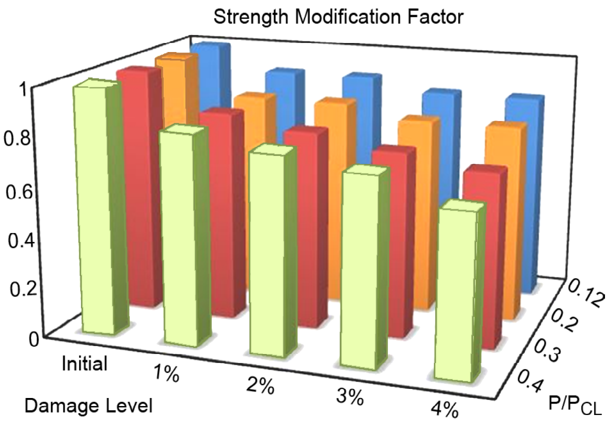

Figure 3. Column strength modification factors corresponding to the peak displacement [13].

3. Designed Steel Frame and Earthquake Ground Motions Considered

3.1. Building Frame Models

A regular 3D special steel moment frame was designed using AISC 360-10 in three bays and four floors. In the model, the length of bays on X and Y axes was 5 m and the floors height was 3.2 m. Figure (4) shows a schematic illustration of the model. It should be mentioned that the designed structure have uniform mass and stiffness distribution over the height.

The designed frame is based on the following assumption, which is commonly adopted in many models with member-by-member representation:

1) Mass is lumped at floors.
2) Concentrated plastic hinges at member ends represent member plasticity.
3) P-Δ effects are considered as the product of the axial force, P, and the story displacement, Δ, divided by the story height is subtracted from the story.
4) A viscous damping of 5% customary for this type of building has been applied to the analyses.

In order to use damaged beam and column plastic hinges from Hosseini Hashemi and Naserpour [13] study, beams and columns members should satisfy the requirement for special ductile members as defined in design codes. Section properties of all frames are depicted in Table (1) [14]. Besides, yield stress is considered to be 240 MPa and ultimate stress is 400 MPa.

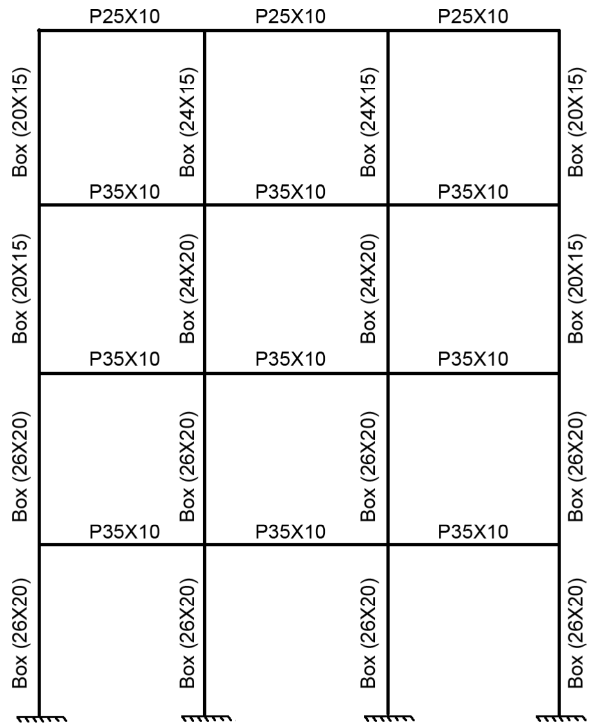

Figure 4. 3D view and the frame of the designed structure.

3.2. Ground Motions

The significant duration of a ground motion at a site depends on various factors, such as earthquake moment magnitude, distance to the fault rupture, depth to the top of rupture, and the soil type of

Table 1. Column and beam sections of the designed structure.

	Columns Sections			Beam Sections	
	BOX 2015	BOX 2420	BOX 2620	P 2510	P 3512
b_f (cm)	20	24	26	12.5	17.5
t_f (cm)	1.5	2	2	1	1.2
d (cm)				25	35
t_w (cm)				0.6	0.6

earthquake [15]. The mainshock earthquake ensembles used in this study are modelled by ground motions developed in the SAC Project for Los Angeles, CA [16]. All the records used in the present study are shown in Table (2).

3.3. Nonlinear Modeling

The seismic response of the building model is evaluated using nonlinear time history analyses. A model comprised by lumped plasticity beam-column elements has been used to represent the flexural behavior of damaged beams and columns. The damaged plastic hinges in the lumped plasticity beam-column elements are modeled by using the damaged hysteretic behavior developed by referenced paper [13], which is capable of simulating stiffness and strength degrading of beams and columns during the repeated earthquakes.

The model is subjected to the ground motions with particular intensities, and the time history of building responses is included key engineering demand parameters such as peak inter-story drifts, roof drifts, and permanent inter-story drifts.

At first, rotations of each plastic hinge of the structure have been determined under those records. Next, based on the deformation of hinges, suitable damaged hinges were assigned to each hinge. At last, the nonlinear time history analyses have been carried out to evaluate the performance of the structure with damaged plastic hinges under repeated earthquakes.

The method used in this paper has been depicted in Figure (5) using a flowchart.

4. Results and Discussion

4.1. Inter-Story Drift

In order to study the influence of mainshock-aftershock earthquake sequences in the response of special steel moment frames, a series of nonlinear dynamic analyses were carried out with considering the effects of initial damage on structural elements.

The distribution over the height of inter-story drift levels computed from a set of the records during the Imperial Valley and Kobe earthquakes for the model is shown in Figure (6).

In these records, it can be seen that in some cases, the mainshocks trigger inter-story drift demands shorter than the aftershock inter-story drift demands. In addition, the effects of cumulative damage on the structure have not been concentrated on some special floors. The results show that in an earthquake, the inter-story drifts are in higher floors

Table 2. Records used for nonlinear time history analyses.

Earthquake	Station	V_s (m/s)	Distance (km) (Joyner-Boore)	Magnitude
Chalfant Valley-02	54171 bishop-LADWP south St	271.4	14.3	6.19
Hector Mine	Amboy	271.4	41.82	7.13
Imperial Valley	5154 EC County Center FF	192.1	7.31	6.53
Kobe	O Shin-Osaka	256	19.14	6.9
Kocaeli, Turkey	Duzce	276	13.6	7.51
Landers	22074 Yermo Fire Station	353.6	23.62	7.3
Superstition Hills (B)	01335 El Centro Imp. Co. Cent	192.1	18.2	6.54

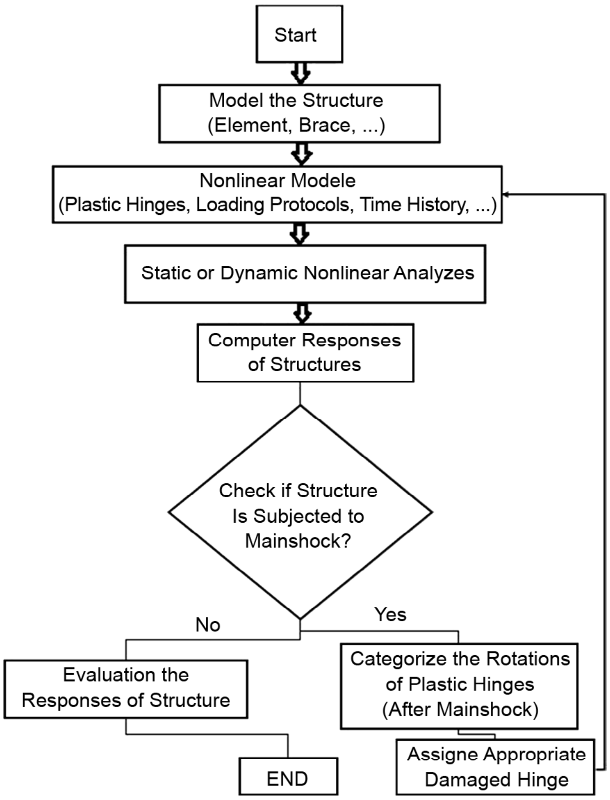

Figure 5. The used method to evaluate structures subjected to mainshock-aftershock sequences.

and in other earthquakes, they are in lower or middle floors. Therefore, it seems that the distribution of inter-story drift levels for damaged structures depends on the elements rotations subjected to mainshocks. It should be mentioned that in previous researches, the seismic behavior of structures subjected to mainshock-aftershock sequences had been evaluated based on the constant shape of components plastic hinge. However, the seismic behavior of damaged structures, moreover, the structure period, the number of story, input strong ground motions, etc. are related to the plastic behavior of the components.

It was observed that the aftershocks in some cases have not increased the level of the inter-story drift and the permanent inter-story drift. For instance, in Superstition Hills earthquake (mentioned in Tables (3) and (4)) in two directions, the distribution of the inter-story drift level in height for the damaged and initial structure is the same.

The rate of the increase in inter-story drifts for each story in two directions has been shown in Tables (3) and (4). The differences among the story

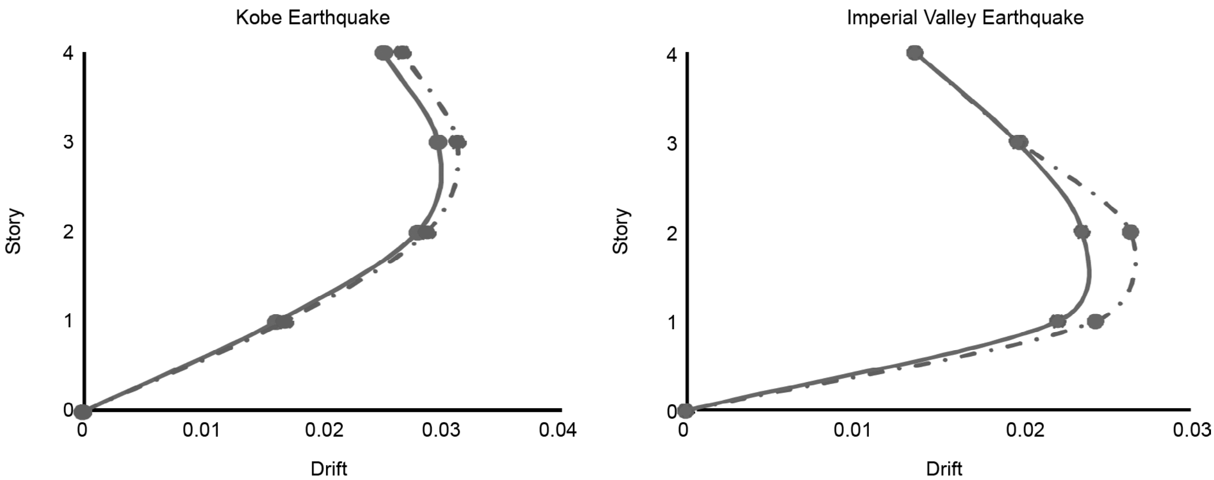

Figure 6. The distribution of inter-story drifts in the height for Kobe and Imperial Valley earthquake.

Table 3. The ratio of the damaged inter-story drift to intact inter-story drift in X-direction

Earthquake	1st Story	2nd Story	3th Story	4th Story
Chalfant Valley-02	0.5	0.5	1	1.1
Hector Mine	2.5	2	4	7
Imperial Valley	0	0	0.5	3
Kobe	4	3	5	6
Kocaeli,Turkey	0	4	8	8.5
Landers	10	7	4	4.5
Superstition Hills (B)	0	0	0.5	0.5

Table 4. The ratio of the damaged inter-story drift to intact inter-story drift in Y-direction.

Earthquake	1st Story	2nd Story	3th Story	4th Story
Chalfant Valley-02	0.5	0.5	1	1
Hector Mine	0.5	0.5	0.5	0.5
Imperial Valley	10	12	0.8	0.2
Kobe	1	1	1.1	1.1
Kocaeli,Turkey	1	0.8	0.5	0.5
Landers	0	0	0.5	0.5
Superstition Hills (B)	0	0	0	0.5

drift of the structure subjected to the earthquakes have been shown in Figures (7) and (8).
The distribution of the inter-story drift for the intact and damaged structure obtained under all referenced records has been illustrated in Figures (9) to (12).

4.2. Permanent Inter-Story Drift

In this chapter, the permanent inter-story drift of the damaged structure has been investigated by NTHA (Nonlinear Time History Analyses). The rate of the permanent inter-story drift in the damaged structure to the intact structure has been shown in Tables (5) and (6) in two direction. It seems that the rate of the permanent inter-story drift in the damaged structure to the intact structure is not related to the rate of the inter-story drift in the damaged structure to the intact structure. For instance, in Landers earthquake, although the rate

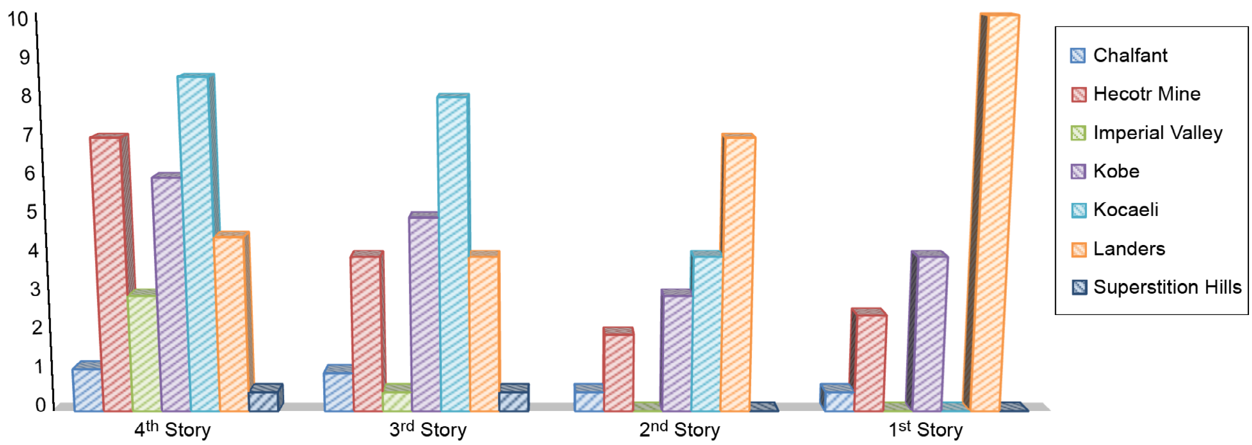

Figure 7. Comparing the ratio of the intact to damaged drifts of the story of the structure in X-direction.

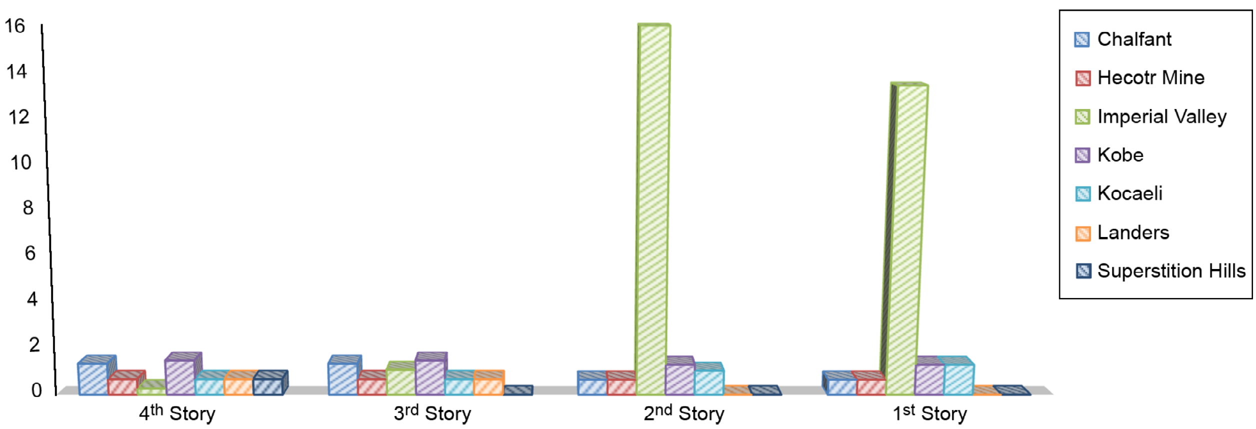

Figure 8. Comparing the ratio of the intact to damaged drifts of the story of the structure in Y-direction.

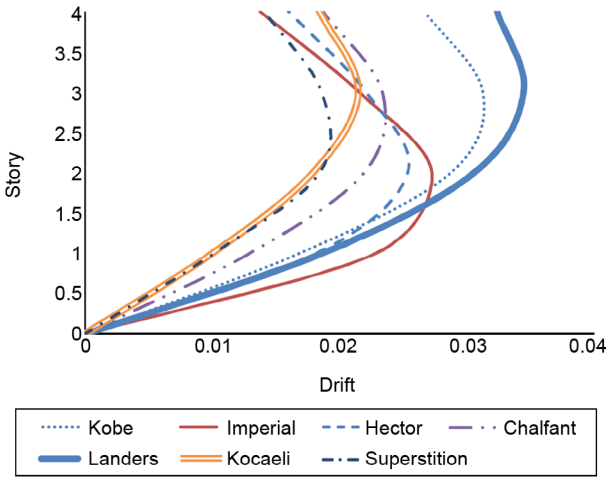

Figure 9. The distribution of inter-story drifts in the height of the damaged structure in X-direction.

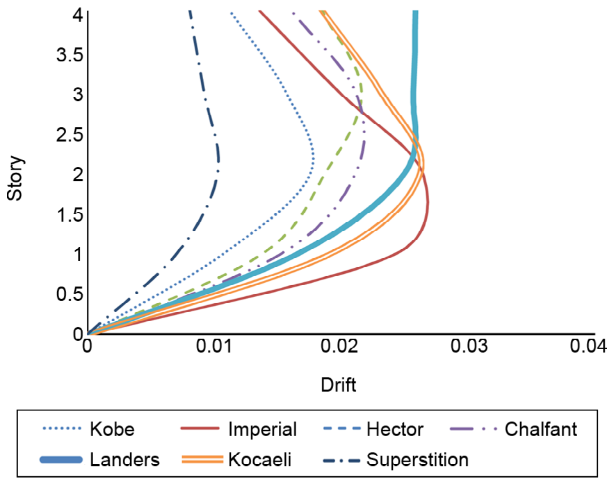

Figure 10. The distribution of inter-story drifts in the height of the damaged structure in Y-direction.

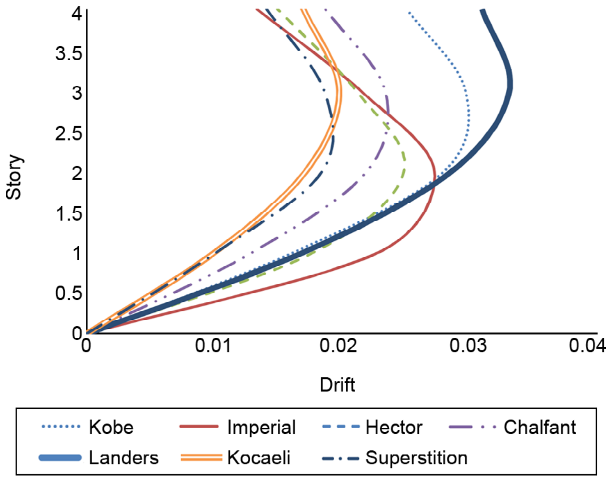

Figure 11. The distribution of inter-story drifts in the height of the intact structure in X-direction.

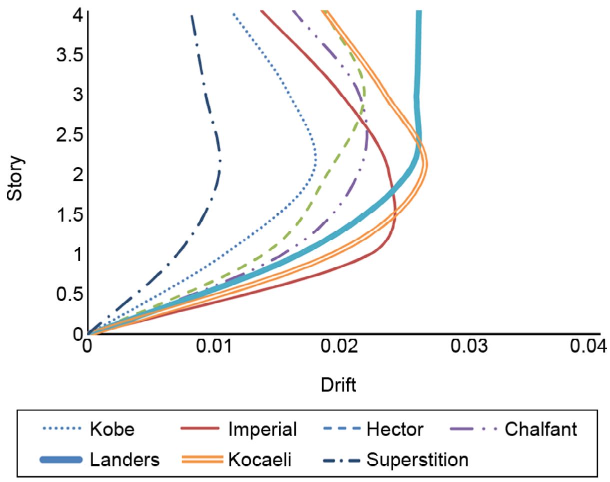

Figure 12. The distribution of inter-story drifts in the height of the intact structure in Y-direction.

Table 5. The ratio of the damaged permanent inter-story drift to intact inter-story drift in X-direction.

Earthquake	1st Story	2nd Story	3th Story	4th Story
Chalfant Valley-02	8	8	8.5	8.5
Hector Mine	11	6	16	17
Imperial Valley	20	40	33	43
Kobe	18	14	13	13
Kocaeli,Turkey	12	12	12	14
Landers	35	37	38	39
Superstition Hills (B)	3	3	3	4

Table 6. The ratio of the damaged permanent inter-story drift to intact inter-story drift in Y-direction.

Earthquake	1st Story	2nd Story	3th Story	4th Story
Chalfant Valley-02	7	7.5	7.5	8
Hector Mine	1	1	1	1
Imperial Valley	14	22	20	27
Kobe	7	7	5	4
Kocaeli,Turkey	7	13	5	10
Landers	13	14	14	14
Superstition Hills (B)	2	2	2.5	3.5

of the inter-story drift of the damaged structure to the intact structure in X-direction can be negligible, the rate of the permanent inter-story drift of damaged structure to the intact structure in X-direction is twice as high as Kobe earthquake. Therefore, it seems that the permanent inter-story drift of damaged steel moment frames does not depend on the general shape of the inter-story drift levels. The permanent drifts of stories of the structure have been compared and illustrated in Figures (13) and (14).

5. Repeated Seismic Sequences

In order to investigate the effects of repeated earthquakes on the structural collapse capacity, an NTHA is carried out using the repeated method to determine the collapse capacity of the damaged building. The repeated method has been evaluated and used in [3, 6, 7, 8]. The strong ground motions in this method is the same as damaged modified hinges method proposed in this paper.

Figure (15) compares the drift ratios of the structure determined by repeated method with damaged modified hinges method.

The results show a fundamental difference between the drifts distribute of the structure in the height computed by damaged modified hinges and the repeated method. The pattern of distributed drift in the height for both methods is not the same. Although, in repeated method for a 4-story building, the ratio of the damaged drift to initial drift is increasing in the height, the rate of this parameter

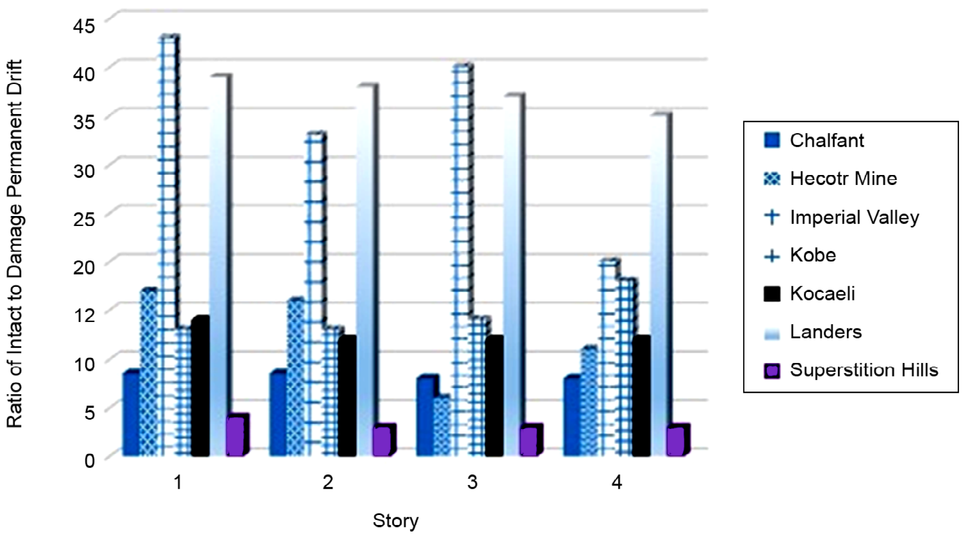

Figure 13. Comparing the ratio of the intact to damaged permanent drifts of the story of the structure in X-direction.

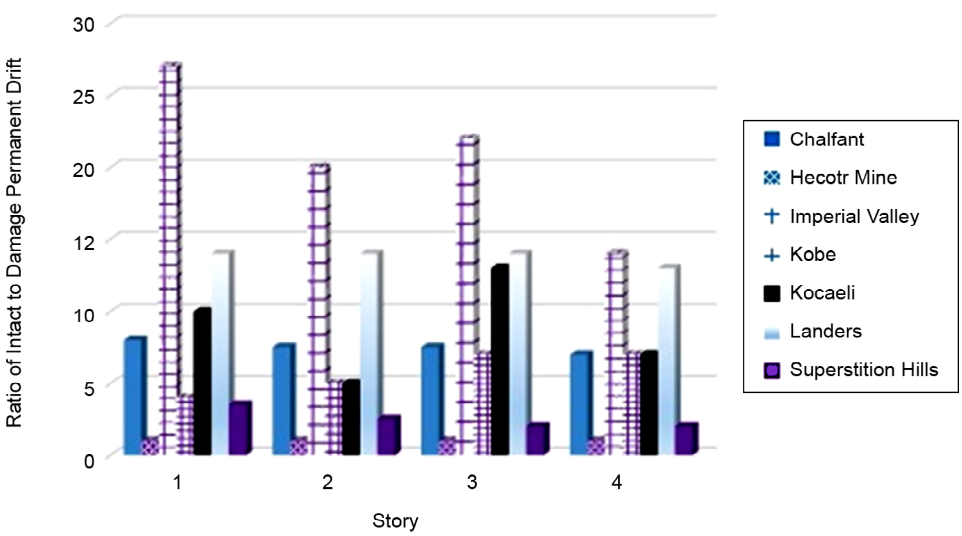

Figure 14. Comparing the ratio of the intact to damaged permanent drifts of the story of the structure in Y-direction.

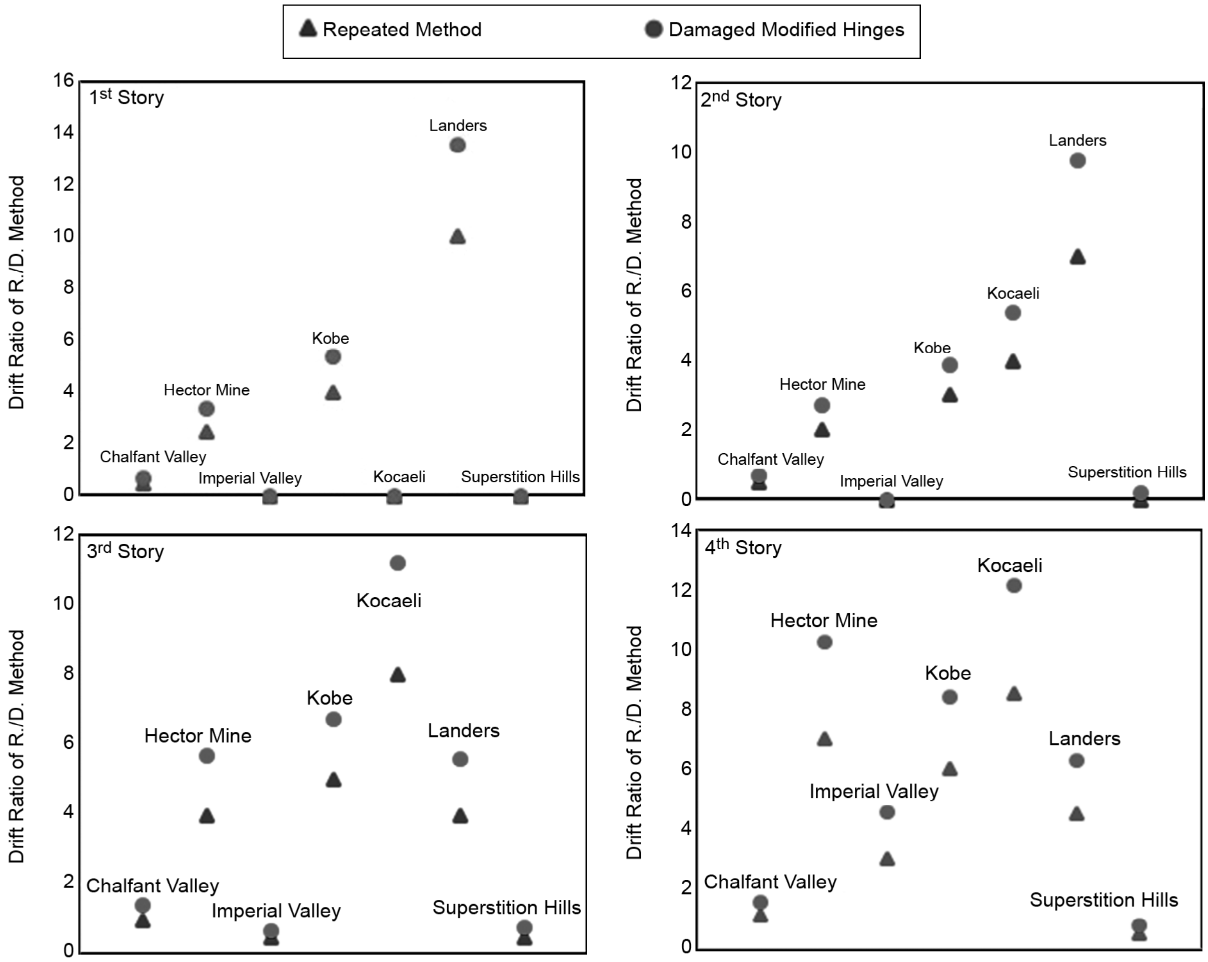

Figure 15. Comparing the Repeated and Damaged Modified Hinges method.

is not as high as the rate of that in the damaged modified hinges.

In the damaged modified hinges method for some strong ground motions, the pattern of damage and the structural mechanism is different from the common mechanism that is the base assumption of design code.

6. Conclusion

This paper has summarized the results of an analytical study aimed at providing further understanding on the influences of the cumulative damage in structural elements on initial and permanent inter-story drift demands in regular low-rise damaged moment-resisting frame buildings under mainshock-aftershock sequences. This study focused on investigating whether considering the effects of degradations in seismic parameters based on the nonlinear dynamic analysis caused by mainshock could change the increase peak (transient) and (residual) permanent drift demands of structures in aftershocks or not. For this purpose, the nonlinear time history analyses have been carried out by a set of records that were recorded in far-field accelerographic stations. At first, the seismic behaviors of the intact structure have been investigated, and the rotations of plastic hinges after mainshock computed and categorized. Then based on the plastic hinges rotation, the suitable damaged hinges were assigned to the structural elements. At last, the behavior of the damaged structure were evaluated by a set of records the same as the initial imposed mainshocks. It can be seen that the inter-story drift of the damaged structure is related to the initial structural components rotations of the structure under mainshocks. In addition, the permanent inter-story drift levels of the damaged structure do not depend on inter-story of the damage structure. Besides, based on the results described above, the intensity and duration of ground motions plays a significant role in the collapse resistance of a structure under repeated earthquake. Therefore, it is suggested that the effects of seismic sequences during performance-based assessment of existing structures should be taken into account by using the effects of degradation of structural element behaviors instead of mainshock-aftershock sequences ground motions.

Finally, the results of Damaged Modified Method have been compared with Repeated Method that is a common method to evaluate the effects of aftershocks on structural collapse capacity. The results show that the Damaged Modified Method is more conservative than Repeated Method. In addition, it is shown that considering the initial damage in element behaviors could change the structural mechanism.

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