

Article

Numerical Simulation on Aftershock Fragility of Low-Ductility RC Frames under Different Mainshock-Induced Damage Conditions

Bomin Qiu ^{1,2}, Jianping Han ^{1,2,*}  and Jun Li ³

¹ Key Laboratory of Disaster Prevention and Mitigation in Civil Engineering of Gansu Province, Lanzhou University of Technology, Lanzhou 730050, China

² Institute of Earthquake Protection and Disaster Mitigation, Lanzhou University of Technology, Lanzhou 730050, China

³ Lanzhou Branch, Changzheng Engineering Co., Ltd., Lanzhou 730050, China

* Correspondence: jphan@lut.edu.cn

Abstract: Aftershocks typically occur multiple times following major earthquakes, potentially inflicting enhanced damage. It is crucial to quantify the impact of aftershocks on the seismic performance of low-ductility reinforced concrete (RC) frames with different mainshock-induced damage states. For this purpose, this study investigated the aftershock fragility of low-ductility RC frames with different damage states induced by mainshocks via the case study of a six-story RC frame without a seismic design. The models of the low-ductility RC frames with different damage states were established via OpenSees software 2.5.0. Incremental dynamic analysis (IDA) was carried out to establish damage states induced by a mainshock. Then sixty-five real aftershocks were inputted to analyze the structures with different main-induced damage states. Aftershock fragility curves of low-ductility RC frame structures with different damage states were obtained. The results show that the exceedance probability of the low-ductility RC frame with mainshock-induced damage is higher than that of the intact low-ductility structure, corresponding to each limit state. The severity of the mainshock-induced damage directly amplifies the demand for a low-ductility RC frame in the subsequent aftershocks. As the mainshock-induced damage increases, the exceedance probability of the low-ductility RC frame undergoing more severe damage under aftershocks significantly increases.

Keywords: low-ductility RC frame; mainshock-induced damage; damage state; aftershock fragility; incremental dynamic analysis; exceedance probability



Citation: Qiu, B.; Han, J.; Li, J.

Numerical Simulation on Aftershock Fragility of Low-Ductility RC Frames under Different Mainshock-Induced Damage Conditions. *Buildings* **2024**, *14*, 1519. <https://doi.org/10.3390/buildings14061519>

Academic Editor: Fabio Di Trapani

Received: 10 April 2024

Revised: 12 May 2024

Accepted: 21 May 2024

Published: 24 May 2024



Copyright: © 2024 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (<https://creativecommons.org/licenses/by/4.0/>).

1. Introduction

Historical seismic events indicate that major earthquakes are often accompanied by numerous aftershocks, such as the 1994 Northridge earthquake [1], 2008 Sichuan Wenchuan earthquake [2], 2011 Tohoku earthquake [3], and 2013 Sichuan Lushan earthquake [4]. For structures damaged during the mainshock, there is often insufficient time for repair and reinforcement before aftershocks occur. The successive aftershocks may cause cumulative structural damage, leading to more severe destruction and potential collapse and ultimately resulting in greater casualties and property losses [5,6]. Therefore, investigating the nonlinear response of structures under aftershock effects is crucial for rapidly evaluating the post-earthquake structures.

Research on the impact of aftershocks on structures began in 1980, with analyzing the nonlinear behavior of single-degree-of-freedom (SDOF) systems under aftershocks recorded during the 1972 Managua earthquake [7]. Subsequently, the impact of aftershocks on structures has been extensively researched, and many studies on the response of single-degree-of-freedom (SDOF) systems under aftershocks have been proposed [8–12].

Aschheim and Black [8] studied the effect of stiffness degradation on the seismic performance of structures under aftershocks. Hatzigeorgiou et al. [11,12] developed a simple and effective method for estimating the nonlinear displacement ratio of a SDOF system under multiple earthquake events, as well as the displacement ductility demand spectrum of the SDOF system under the mainshock–aftershock sequence. However, the applicability of the SDOF system has limitation owing to its idealization and simplification. It cannot accurately replicate the actual response of structures under aftershock effects.

To simulate the actual response of structures under an earthquake more accurately, multi-degree-of-freedom (MDOF) systems can be utilized to capture the localized damage in different components of a structure under aftershock effects. Hatzigeorgiou and Liolios [13] employed MDOF systems to study the inter-story drift, maximum displacement, and plastic hinges of the RC structural parameters under mainshock–aftershock sequences. Goda and Salami [14] investigated the aftershock effects on the seismic fragility of a conventional wood frame using a large number of mainshock–aftershock sequences. Fragiocomo et al. [15] analyzed the damage response of structures under mainshock–aftershock sequences. Tesfamariam et al. [16] compared the seismic responses of a bare RC frame, a masonry-infilled RC frame with an open ground story, and a fully infilled RC frame when subjected to a mainshock–aftershock sequence. Han et al. [17] proposed a novel method for synthesizing mainshock–aftershock sequences, which was used to assess the seismic performance of buildings considering aftershocks.

In addition, fragility analysis, as an essential method for assessing the safety performance of structures, is applied to various structural systems under aftershock effects, such as RC buildings [18–20], steel plate shear wall structures [21], wood frame [14,22], and self-centering frame [23–26]. However, previous studies on mainshock–aftershock sequences have primarily concentrated on new structures and various building types, with older buildings receiving comparatively less attention. A significant number of low-ductility reinforced concrete (RC) buildings still exist in western China, and they are vulnerable to brittle damage due to inadequate seismic reinforcement, lack of seismic construction measures, and inferior material properties. Furthermore, buildings that have been in use for decades demonstrate characteristics like reduced ductility, poor energy dissipation, and weakened resistance to collapse. Consequently, these low-ductility structures are more prone to the effects of aftershocks. Only a few studies have been conducted on the seismic analysis of low-ductility structures considering the mainshock–aftershock effects [27–29].

Low-ductility structures are susceptible to brittle failure during seismic events, often manifesting in failure modes like node shear failure, column bending shear failure, and bar slip at beam ends. Therefore, it is a notable challenge to accurately simulate the above three deformation components in low-ductility structures using finite element analysis software and to further evaluate the fragility of low-ductility structures during aftershocks particularly considering diverse mainshock-induced damage states [30–32]. Elwood [33] proposed that the method of using a shear spring attached to the end of a common fiber-based, nonlinear beam–column element can effectively simulate the coupling effect of the axial–shear–bending response. Jeon [28] developed the method of using a rotation spring to simulate the shear deformation of the beam–column joint zone. d’Aragona [27] studied the aftershock fragilities conditioned on the return period (intensity) of the damaging ground motion.

This paper introduces a probabilistic approach for quantifying the aftershock fragility analysis of low-ductility RC frames with different mainshock-induced damage states. First, a model was developed, which incorporated a shear spring at the end of the fiber-based nonlinear beam–column element to account for the flexure–shear interaction. Additionally, a rotational spring was placed at the joint to represent the shear deformation within the beam–column joint zone. Second, the IDA method was adopted to establish the mainshock-induced damage states by a mainshock. Finally, the aftershock fragility of low-ductility RC frames under various mainshock damage states was analyzed by employing the cloud method [34], using 65 actual aftershock ground motion records.

This study aims to provide a theoretical basis for post-earthquake strengthening and decision-making processes associated with post-earthquake rescue efforts for low-ductility structures.

2. Aftershock Fragility Analysis on Low-Ductility Structures with Mainshock-Induced Damage

2.1. Establishment of Mainshock-Induced Damage Levels

It is a crucial step to establish different mainshock-induced damage states (IDS_i) in the aftershock fragility analysis of low-ductility RC frames. The damage stages of structures induced by a mainshock can be determined by incremental dynamic analysis (IDA). The damage state in the mainshock simulation is primarily governed by the maximum inter-story drift ratio of a structure. The fundamental principle of IDA simulation approaches is described as follows.

The IDA-based approach simulates the different damage states of low-ductility RC structures by IDA under a mainshock. The specified mainshock damage state is accomplished through adjusting the intensity measure of a selected mainshock ground motion. Initially, IDA is performed on the structure to simulate different mainshock-induced damage states by scaling mainshock ground motion. Subsequently, different aftershock ground motions are inputted to evaluate the aftershock fragility of the structure characterized by different mainshock damage states, as shown in Figure 1. The IDA approach evaluates the seismic performance of a structure by progressively increasing the intensity measure of ground motion and observing the response of structures at various intensities. This approach can accurately reflect the realistic response of the structure under earthquake effects and the damage state of the structure under the mainshock.

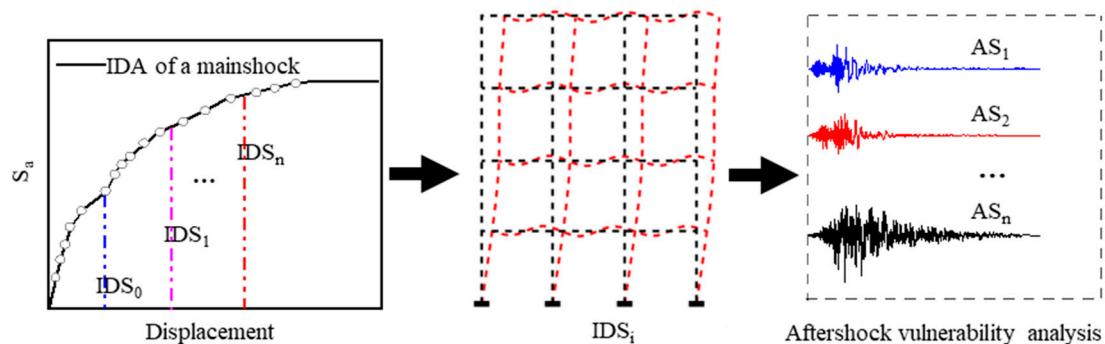


Figure 1. Analysis of different mainshock damage states based on the IDA method (AS_i —aftershock ground motion).

Therefore, this study adopted the IDA method to establish the mainshock-induced damage states, aiming to investigate the aftershock fragility of low-ductility RC frames under various mainshock damage states. A mainshock ground motion is used to determine the initial damage of a structure. Then, a series of actual aftershock ground motions are used for time-history analysis by employing the cloud method [34].

2.2. Aftershock Fragility Analysis of Low-Ductility Structures

Fragility analysis, which is a core aspect of next-generation performance-based earthquake engineering (PBEE), can be used to predict the conditional probability of structural damage under different earthquake intensities. In this study, the spectral acceleration (S_a) is used as the ground motion intensity index. The traditional fragility function [35] is defined as follows:

$$P\{D > C|S_a\} = \phi \left[\frac{\ln(m_D/m_C)}{\sqrt{\beta_{D|S_a}^2 + \beta_C^2 + \beta_M^2}} \right] \quad (1)$$

where $P\{D > C|S_a\}$ is the probability that a structure will experience or exceed a specified level of damage for a given ground intensity measure ($IM = S_a$). m_D is the median value of the seismic demand at $IM = S_a$, m_C is the median value of the structural capacity, and $\beta_{D|IM}$ and β_C are the logarithmic standard deviation of the seismic demand and structural capacity, respectively. β_M denotes the uncertainty of the model and is assumed to be 0.2 [35].

The fragility analysis of low-ductility structures for different damage states associated with the mainshock is based on Equation (1). In contrast with the traditional fragility analysis, the spectral acceleration of aftershocks ($S_{a,AS}$) corresponding to the basic period T_1 is used as an aftershock intensity index. A statistical correlation between $S_{a,AS}$ and seismic demand D_{MA} is established for the aftershocks of low-ductility structures with different damage states after the mainshock, as delineated below:

$$\ln m_{D_{MA}|S_{a,AS}} = a + b \ln S_{a,AS} \quad (2)$$

$$\beta_{D_{MA}|S_{a,AS}} = \sqrt{\frac{\left(\sum \ln D_{MA,i} - \ln D_{MA}|S_{a,AS}\right)^2}{N - 2}} \quad (3)$$

where a and b are regression coefficients, $m_{D_{MA}|S_{a,AS}}$ is the median value of the seismic demand (D_{MA}) under the aftershock intensity ($S_{a,AS}$), $\beta_{D_{MA}|S_{a,AS}}$ denotes the logarithmic standard deviation of the demand corresponding to the aftershock intensity $S_{a,AS}$, $D_{MA,i}$ corresponds to the seismic demand under the i th aftershock, and N is the number of aftershock sequences used in the probabilistic seismic demand analysis. The aftershock fragility function for low-ductility structures with different mainshock-induced damage states can then be expressed as follows:

$$P\{D > C|S_{a,AS}\} = \phi \left[\frac{\ln m_{D_{MA}|S_{a,AS}} - \ln m_C}{\sqrt{\beta_{D_{MA}|S_{a,AS}}^2 + \beta_C^2 + \beta_M^2}} \right] \quad (4)$$

where $P\{D > C|S_{a,AS}\}$ denotes the probability of exceeding a certain limit state for low-ductility structures with different mainshock-induced damage states under an aftershock intensity ($S_{a,AS}$).

The steps for executing aftershock fragility analysis on low-ductility structures can be summarized as follows:

- 1) Establishment of a structural analysis model. This involves formulating an analysis model for low-ductility structures, considering failure mechanisms such as shear failure in the beam–column joint zone, column shear failure, and bond-slip failure of longitudinal reinforcement.
- 2) Simulation of the initial damaged structure. This process involves using a mainshock to define mainshock damage states of low-ductility structures by the IDA approach.
- 3) A nonlinear time history analysis of structures with different mainshock-induced damage states during the main shock under an aftershock was performed.
- 4) Determine aftershock demand parameters and limit states. The cloud method [34] was employed to establish a structural response cloud map for the structure that experienced aftershocks under various mainshock-induced damage conditions. The probabilistic seismic demand parameters under an aftershock (a , b , and $\beta_{D_{MA}|S_{a,AS}}$) were derived from Equations (2) and (3).
- 5) Establishment of aftershock fragility curves for low-ductility structures corresponding to different mainshock-induced damage states. The aftershock fragility curves of the low-ductility structures corresponding to different damage states were established from the obtained probabilistic seismic demand parameters (a , b , and $\beta_{D_{MA}|S_{a,AS}}$), combined with Equation (4).

Based on the above analysis steps, the aftershock fragility analysis method for low-ductility structures with different damage states resulting from the mainshock can be visually depicted as illustrated in Figure 2.

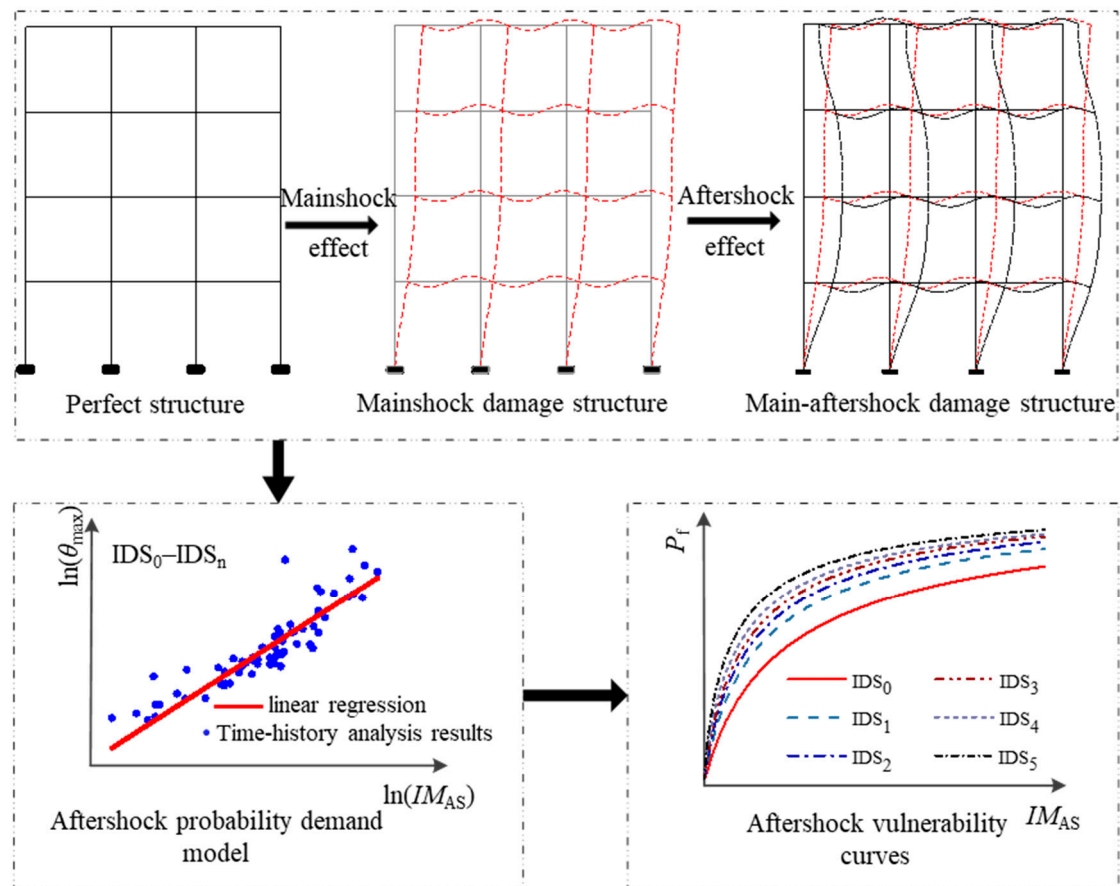


Figure 2. Aftershock fragility analysis of low-ductility structures under different damage conditions of mainshock.

3. Low-Ductility Structure and Numerical Model

3.1. Prototype Structure

In this study, the prototype building is a specific RC frame in the northwest region of China, which was designed only considering gravity loads, without any reference to seismic fortification requirements. The building has a total height of 21 m and a floor height of 3.3 m for most floors, except for the first floor, which has a height of 4.5 m. The plan, elevation, and reinforcement diagrams of the structure are shown in Figure 3. The beams and columns are reinforced with HRB335 hot-rolled steel bars, and the remaining steel bars are reinforced with HPB235. The concrete grade is C30. The beam is 250 mm in sectional width and 500 mm in sectional height, the column section size is 500 mm × 500 mm, and the stirrup specifications are denoted as $\phi 6 @ 200$.

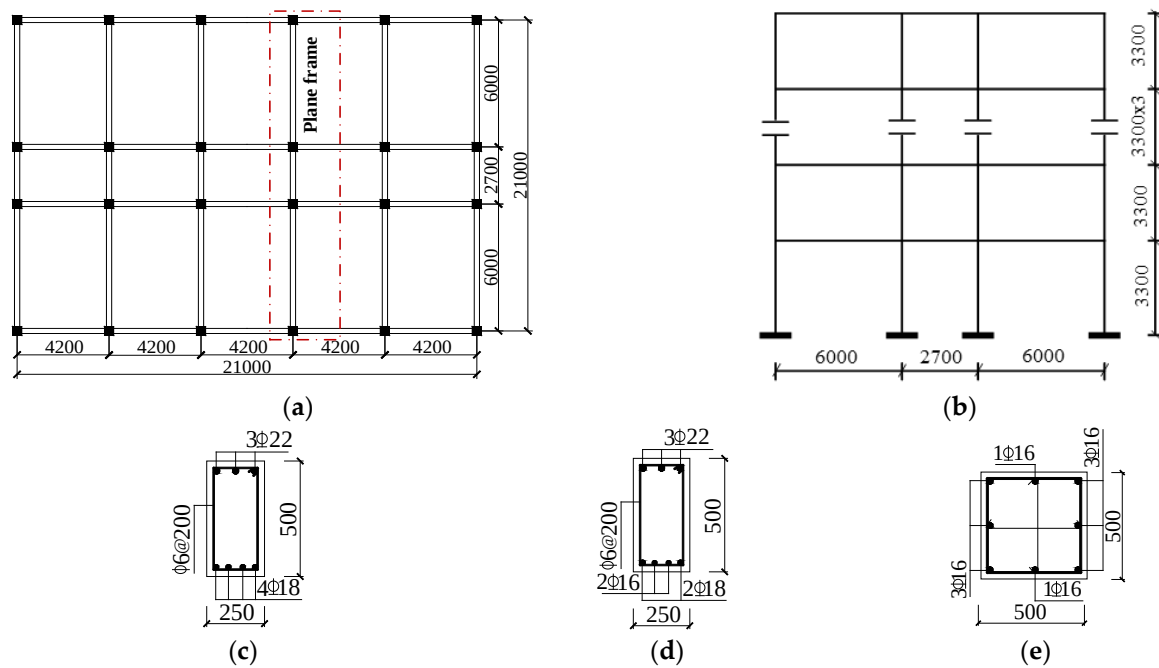


Figure 3. Prototype structure (units: mm): (a) plane drawing, (b) elevation drawing, (c) mid-span beam section, (d) side-span beam section, and (e) column cross section.

3.2. Low-Ductility Structural Model

The shear deformation of the column and the shear deformation failure of the beam–column joint are primary failure modes of low-ductility RC structures during earthquake events. In the present studies, the fiber-based nonlinear beam–column element is used, which only considers bending deformation and cannot simulate shear failure modes. The modified model considering the shear deformation is illustrated in Figure 4. To model the shear deformation, the shear spring at the end of the column and the rotation spring placed in the joint zone are used to simulate the shear deformation of the column and the shear deformation in the beam–column joint, respectively. The fiber-based nonlinear beam–column element is used to define the bending deformation.

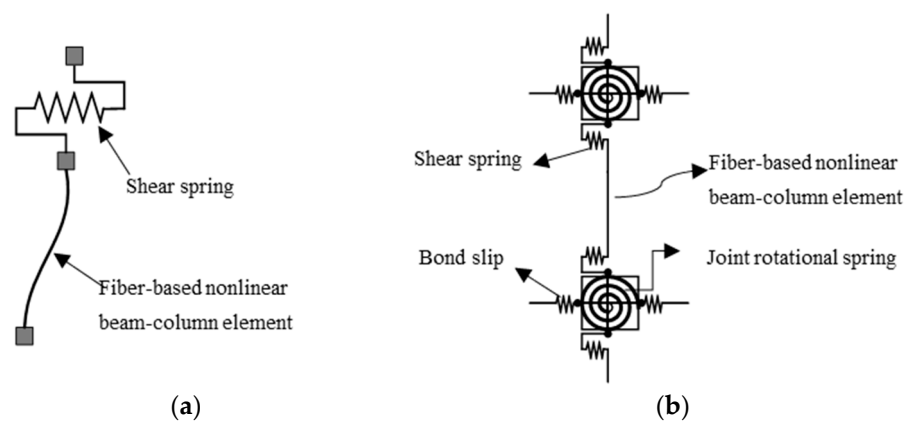


Figure 4. Model method of shear behavior: (a) model method of shear behavior of column [33]; (b) model method of shear behavior of beam–column joint [28].

The flexure–shear column model was developed by using the uniaxial hysteretic material in OpenSees. In Figure 5, V is the shear force of the column end and Δ represents the total deformation caused by bending and shear. As shown in the Figure 5a, the bending and shear deformation of the column is simulated by the deformation of the shear spring at the column end copulating with the bending deformation of the fiber-based beam–column

element. When the shear force (V) generated by external loads is less than the shear bearing capacity of the column (V_n), the bending deformation is the main deformation of the column. If V is greater than V_n , the deformation of the column mainly includes the shear deformation.

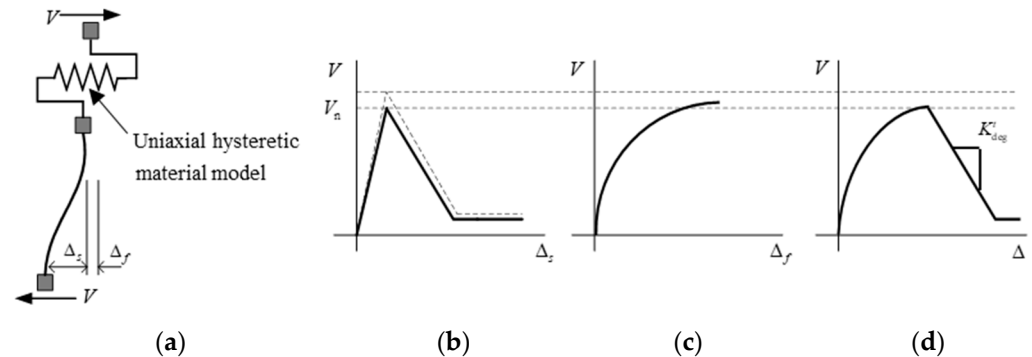


Figure 5. Shear spring based on the uniaxial hysteretic material model [33]: (a) model of shear-bending interaction, (b) shear spring deformation, (c) beam–column deformation, and (d) total response.

The shear bearing capacity of the column is presented as the following Equation (5) [28]:

$$V_n = \frac{A_{sv}f_{yv}h_0}{s} + \left(\frac{0.5\sqrt{f'_c}}{a/h_0} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c}A_g}} \right) 0.8A_g \quad (5)$$

where V_n is the shear bearing capacity of the RC columns, A_{sv} is the area of transverse stirrups, f_{yv} is the yield strength of transverse stirrups, h_0 is the effective height of a column cross section, s is the stirrup spacing, a is the shear deformation span of the RC column, f'_c is the axial compressive strength of concrete, P is the axial force loaded on the column, and A_g is the total area of the column section.

The shear stiffness degradation rate is defined as the following Equation (6) [28]:

$$K_{deg}^t = -4.5P \left(4.6 \frac{A_{sv}f_{yv}d_c}{P_s} + 1 \right)^2 \frac{L}{P_s} \quad (6)$$

where d_c is the depth of the column core from the centerline to the centerline of transverse reinforcement, and L is the column length.

Figure 6 shows the Pinching4 material model [36], which is used to model the rotational spring simulating the shear deformation of the joint zone. The material model can be defined by several key components, including a response backbone, an unload–reload path, and three specific damage rules: unloading stiffness degradation, reloading stiffness degradation, and strength degradation, as shown in Figure 6. The parameters of the eight control points on the skeleton curve are calculated using the simplified Strut and Tie Model (STM) described in reference [37], while the remaining parameters are determined based on references [38,39].

Based on the documented damage mechanisms of the low-ductility structural members suggested by Jeon et al. [28], an OpenSees [40] finite element analysis model was developed. As illustrated in Figure 7, the shear failure of the beam–column joint region, the column shear failure, and the bond-slip failure of the beam’s longitudinal reinforcement were considered for this model. The beam and column were modeled using a nonlinear beam–column element, which captured the distributed nonlinear behavior through fiber sections. The concrete and steel fibers were designated as concrete01 material and steel02 material, respectively. The concrete01 material adopted the Kent–Scott–Park model, which considered the stiffness degradation and restraining effect of hoop reinforcement. The standard unconfined concrete strength was $f_{ck} = 20.1$ MPa. The steel02 material was based

on the Giuffrè–Menegotto–Pinto model with isotropic strain hardening, considering the Bauschinger effect. Corresponding to HRB335 and HPB235, the yield strength of steel was 335 MPa and 235 MPa respectively. The shear deformation in the beam–column joint zone was simulated using a rotation spring, which employed a trilinear Pinching4 material model that incorporates strength degradation, stiffness degradation, and pinching effect. The zero-length element was used to define the shear spring assigned by the uniaxial hysteretic material, as shown in Equation (5), in order to reflect the bending shear failure, strength degradation, and stiffness degradation of the column. The bond slip of the longitudinal steel bar at the beam–node connection was simulated using zero-length elements assigned with the Bond_SP01 material [41] to establish the bond slip of the reinforcement.

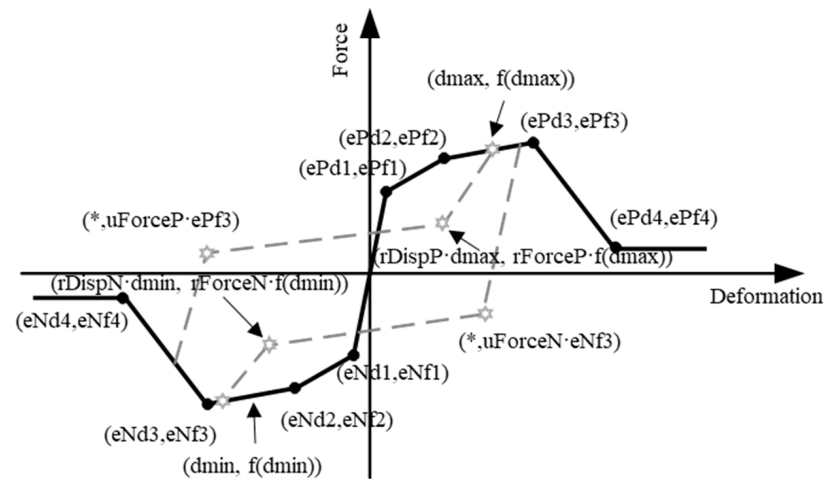


Figure 6. Pinching4 material model [36].

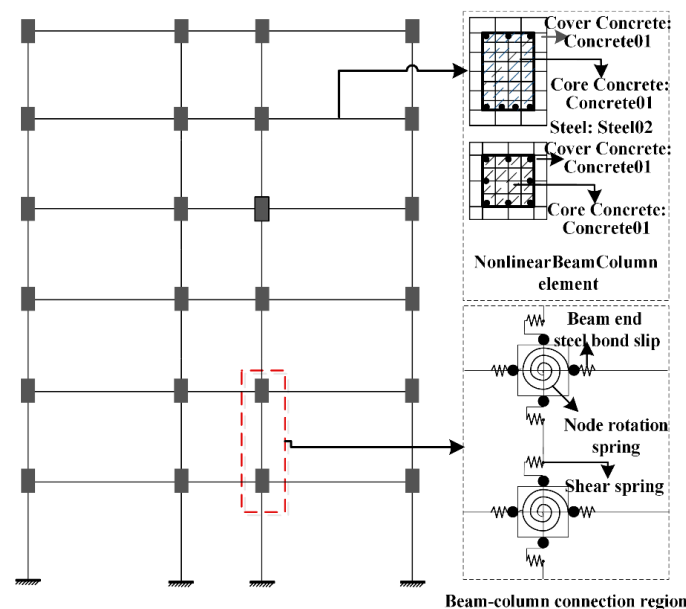


Figure 7. OpenSees numerical model for the low-ductility RC frame.

4. Aftershock Ground Motion Records

4.1. Selection of Aftershock Ground Motion

To perform an aftershock fragility analysis of structures with different mainshock-induced damage states, 65 actual aftershock ground motion records shown in Figure 8 were selected from the Pacific Earthquake Engineering Research Center (PEER) NGA-West2 database [42] according to the principle proposed by Goda and Taylor [43]. All aftershock ground motions have a moment magnitude of 5 or greater, a horizontal seismic peak acceleration (PGA) of 0.04 g or above, a horizontal seismic peak velocity (PGV) of 1.0 cm/s or above, and an average shear wave velocity (V_{S30}) ranging from 100 m/s to 1000 m/s at a depth of 30 m.

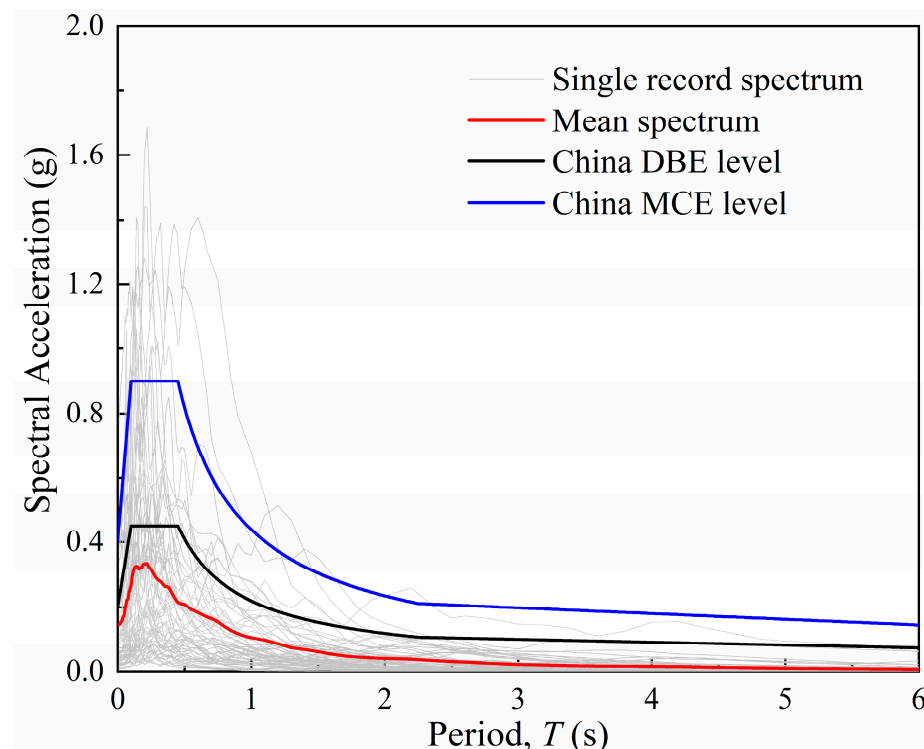


Figure 8. Acceleration spectra of selected ground motion records and the China design spectra.

4.2. Analysis of Aftershock Polarity

The aftershock polarity refers to the positive and negative directions of aftershocks with respect to the mainshock, as shown in Figure 9. Previous studies have demonstrated that the polarity of aftershocks can either magnify or diminish the residual deformation in structures caused by the mainshock [44]. Moreover, as the mainshock-induced damage increases, the influence of the aftershock polarity on the residual deformation increases correspondingly [44]. The structural fragility can be affected significantly by the aftershock directivities. Therefore, it is crucial to consider the polarity of the aftershocks when evaluating the aftershock fragility of low-ductility structures with different mainshock-induced damage states.

A nonlinear time-history analysis was conducted on a low-ductility structure using an actual mainshock–aftershock sequence. The inter-story drift ratio was used to evaluate the structural response. The time-history curves of the inter-story drift ratio depicted in Figure 10 demonstrate a substantial difference in the influence of aftershocks in the positive and negative directions on the structural response following the mainshock. Specifically, aftershock polarity may affect the residual deformation of structures when a mainshock is experienced.

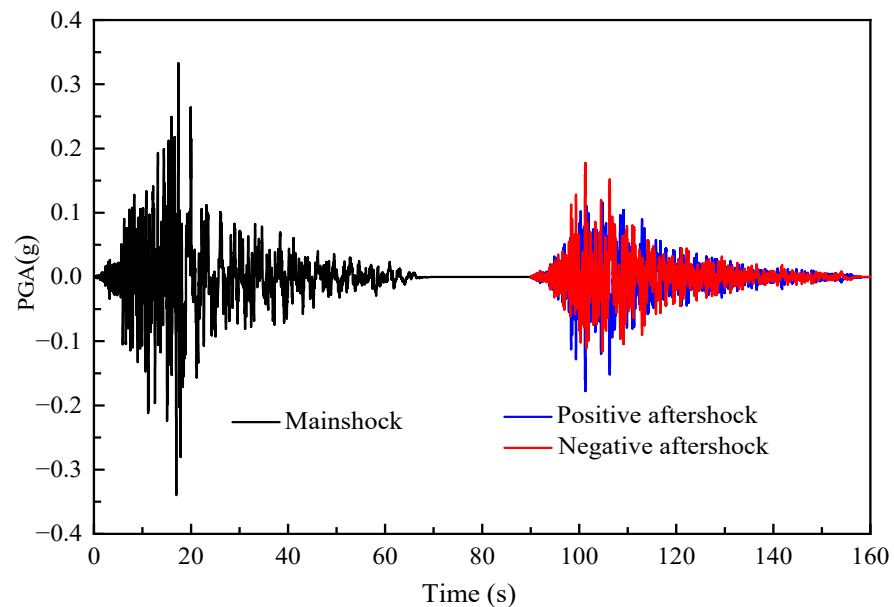


Figure 9. Schematic diagram of the polarity of aftershock ground motion.

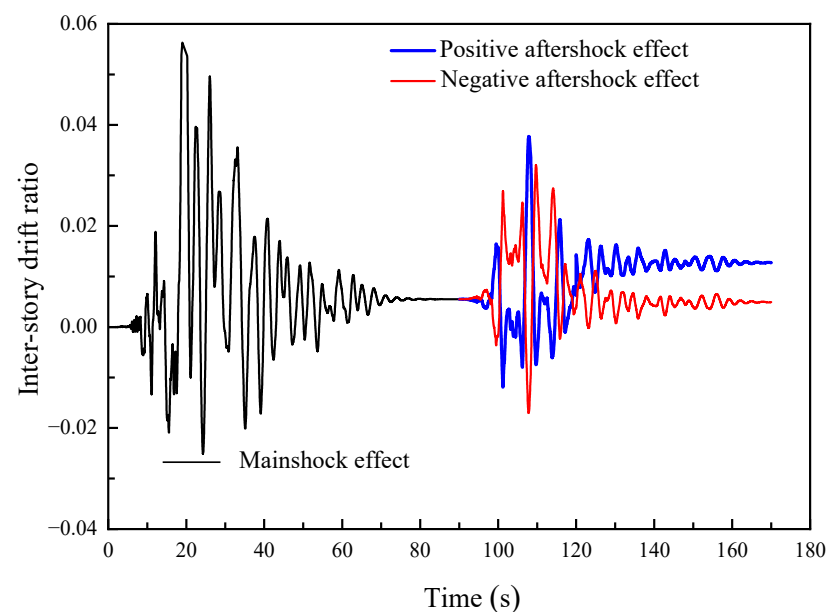


Figure 10. Effect of aftershocks in different directions on low-ductility structure under mainshock-induced damage.

To accurately assess the aftershock fragility of low-ductility structures under different mainshock-induced damage states and avoid the impact of aftershock polarity on the analysis results, the positive and negative aftershocks were considered for low-ductility structures with different mainshock-induced damage when analyzing their aftershock fragility. An aftershock fragility analysis was used to evaluate the mechanical behavior of structures under different damage states caused by the mainshock; a larger value was selected as the representative aftershock response.

5. Aftershock Fragility Analysis

5.1. Classification of Different Mainshock-Induced Damage States

This section analyzed the aftershock vulnerability of a six-story low-ductility structure under different mainshock-induced damage states and discussed the influence of various mainshock-induced damage states on the aftershock fragility of the structure.

The maximum inter-story displacement ratio ($\theta_{\max}$) of a structure has been demonstrated as an effective indicator for assessing the structural damage level in terms of collapse resistance, component failure, and joint rotation capacity [45]. Therefore, the structural damage caused by the main earthquake was classified by the maximum inter-story drift ratio. To study the aftershock fragility of the structure under different mainshock-induced damage states (IDS_i), the different mainshock-induced damage states (IDS_i) of the structure were divided into six classes, as listed in Table 1.

Table 1. Division of different mainshock-induced damage states of low-ductility structures.

Mainshock-Induced Damage State (IDS_i)	IDS_0	IDS_1	IDS_2	IDS_3	IDS_4	IDS_5
Maximum inter-story drift ratio ($\theta_{\max,ms}$)	0	0.005	0.01	0.02	0.03	0.04

The dynamic time-history analysis of the structure against the IDS_0 state (no damage occurring to the intact structure under the main earthquake) was performed solely under an aftershock. To simulate the structure in the IDS_1 – IDS_5 states (representing different mainshock-induced damage levels), a mainshock IDA was initially performed on the structure, followed by an aftershock fragility analysis of the structure with various mainshock damage states.

5.2. Aftershock Probabilistic Seismic Demand Models

IDA was employed to determine the mainshock-induced damage corresponding to the structural state index of IDS_1 – IDS_5 . Then a nonlinear time-history analysis was conducted on the structure with different mainshock-induced damage states under aftershocks. Based on $\theta_{\max}$, the aftershock probabilistic seismic demand model for the frame was computed for each mainshock-induced damage state (IDS_i , $i = 0, \dots, 5$) shown in Figure 11. These results indicate that the seismic demand of the structure with different mainshock-induced damage states under aftershocks and an intensity index ($S_{a,AS}$) have log-linear distributions. Initially, the aftershock probabilistic seismic demand models for the structure in various mainshock-induced damage states were obtained through the linear regression of $(\ln(\theta_{\max}) - \ln(S_{a,AS}))$. Based on the regression findings, the corresponding parameters a , b , and $\beta_{D|S_{a,AS}}$ of the aftershock probabilistic seismic demand were calculated. The seismic demands and their parameters are presented in Table 2.

Figure 11 and Table 2 show that the demand for the mainshock-damaged structure is higher than that for the intact structure. Nonetheless, as the mainshock-induced damage increases, the trend of the seismic demand for low-ductility structures under aftershocks is not evident.

Table 2. Aftershock probabilistic seismic demand parameters for low-ductility structures with different mainshock damage states (IDS_i).

Mainshock Damage States	Aftershock Probabilistic Seismic Demand Model	R^2	$\beta_{D S_{a,AS}}$
IDS_0	$\ln(\theta_{\max}) = 0.698 \times \ln(S_{a,AS}) - 3.735$	0.845	0.463
IDS_1	$\ln(\theta_{\max}) = 0.697 \times \ln(S_{a,AS}) - 3.443$	0.812	0.520
IDS_2	$\ln(\theta_{\max}) = 0.693 \times \ln(S_{a,AS}) - 3.316$	0.825	0.495
IDS_3	$\ln(\theta_{\max}) = 0.676 \times \ln(S_{a,AS}) - 3.242$	0.857	0.428
IDS_4	$\ln(\theta_{\max}) = 0.642 \times \ln(S_{a,AS}) - 3.175$	0.803	0.493
IDS_5	$\ln(\theta_{\max}) = 0.577 \times \ln(S_{a,AS}) - 3.103$	0.751	0.515

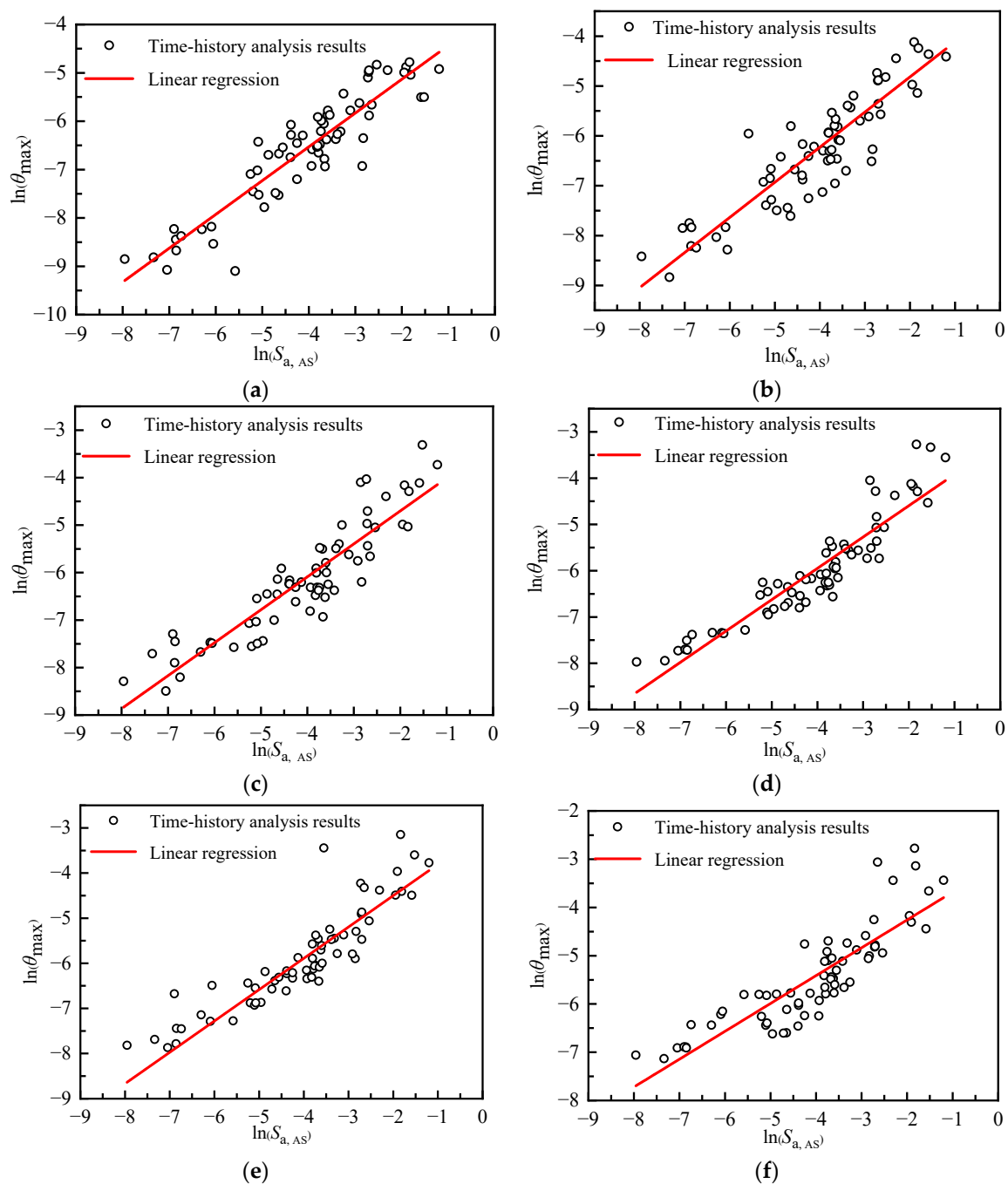


Figure 11. Median fitting values of seismic demand for low-ductility structures with different mainshock damage: (a) IDS_0 , (b) IDS_1 , (c) IDS_2 , (d) IDS_3 , (e) IDS_4 , and (f) IDS_5 .

The analysis results of the aftershock probabilistic seismic demand for the low-ductility structure with different mainshock damage states (IDS_0 – IDS_5) are presented and compared in Figure 12. As the mainshock-induced damage levels increase from IDS_0 to IDS_5 , the seismic demand under aftershocks increases. When the mainshock-induced damage state is IDS_5 , the seismic demand is the highest, and the increase in seismic demand is insignificant when the mainshock-induced damage state changes from IDS_1 to IDS_4 . With an increase in mainshock-induced damage in low-ductility structures, the corresponding seismic demand increases under aftershocks.

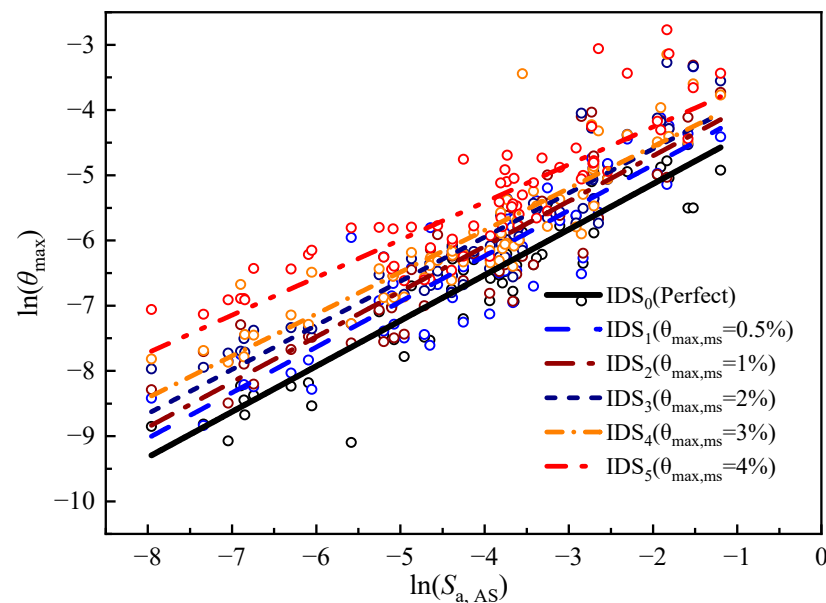


Figure 12. Comparison of aftershock probabilistic seismic demand models for low-ductility structures with different damage states.

5.3. Aftershock Fragility Curves of the Mainshock-Damaged Structure

In the aftershock fragility analysis of structures, the limit states of structures under earthquakes are generally classified into several levels. In this study, four limit states are selected for the low-ductility structure under aftershock action, which are light, moderate, severe, and collapse, as presented in Table 3 [46]. Additionally, Table 3 shows the definitions of the median m_C and logarithmic standard deviation β_C of the seismic capacity for the four limit states [46].

Table 3. Limit states and the corresponding capacity parameters.

Capability Parameters	Limit States			
	Light	Moderate	Severe	Collapse
m_C	0.182% (1/550)	1%	2%	4%
β_C	0.2	0.3	0.3	0.4

Based on Table 3 and Equation (4), the aftershock fragility curves for the low-ductility structures with different mainshock-induced damage states (IDS_i) across the four limit states are plotted in Figure 13. It is observed that larger limit states have fragility curves for larger limit states, such as severe limit state, located at lower positions compared with smaller limit states, such as light limit state, under the same mainshock-induced damage states (e.g., Figure 13b IDS_1). For a low-ductility structure with the same mainshock damage state, the failure probability decreases as the limit state intensifies when it is subjected to aftershock action again. However, the results cannot intuitively reflect the difference in the fragility curves of low-ductility structures with different mainshock-induced damage states (IDS_i) for the same limit state.

According to FEMA695, the spectral acceleration corresponding to a 50% exceedance probability of the structure (denoted as m_R) was defined as the structural collapse resistance capability, which can be obtained from Figure 13 and is shown in Table 4. It can be observed from Table 4 that the collapse resistance capacity of the structure gradually decreased as the mainshock damage state increased. For instance, for the structure in the IDS_2 state, m_R is 1.151 g, which is 45.5% less than that of the structure in the IDS_0 state and 40.4% larger compared with the structure in the IDS_5 state. From the above analysis, it can be concluded

that considering the impact of mainshock damage is crucial for the vulnerability analysis of aftershock effects.

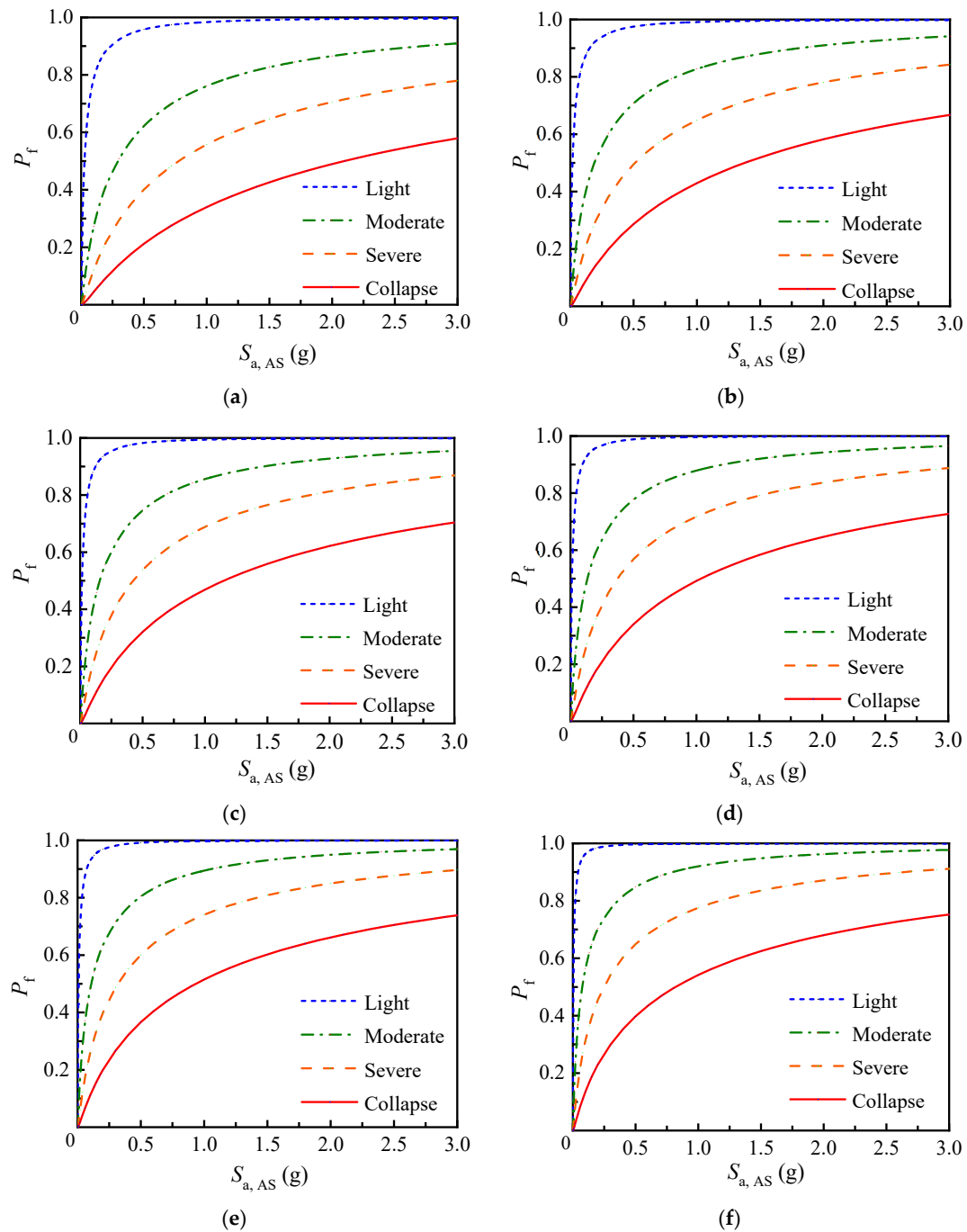


Figure 13. Fragility curves of low-ductility structures with different mainshock damage for aftershock records: (a) IDS_0 , (b) IDS_1 , (c) IDS_2 , (d) IDS_3 , (e) IDS_4 , and (f) IDS_5 .

Table 4. The structural collapse resistance capabilities.

Mainshock Damage States	IDS_0	IDS_1	IDS_2	IDS_3	IDS_4	IDS_5
m_R (g)	2.096	1.379	1.151	1.037	0.935	0.820

To better illustrate the influence of mainshock damage on aftershock fragility, Figure 14 compares failure probability curves corresponding to low ductility structures with different

mainshock damages across four limit states: slight, moderate, severe, and collapse. For example, Figure 14a represents the impact of mainshock damage on the aftershock failure probability of the structure in the light limit state, and Figure 14d depicts the impact of mainshock damage on the aftershock failure probability of the structure when the structure approaches or surpasses the collapse limit state. The results demonstrate that the damage induced by the mainshock affects the aftershock fragility curves. For instance, for a given limit state, the intact structure has a lower failure probability than the mainshock-damaged structures at the given aftershock ground intensity measure ($S_{a,AS}$). The mainshock-induced damage can increase the exceedance probability of low-ductility structures corresponding to various limit states during aftershocks. This trend becomes more evident as the mainshock-induced damage increases, indicating a reduction in the seismic performance of the low-ductility structures with mainshock-induced damage.

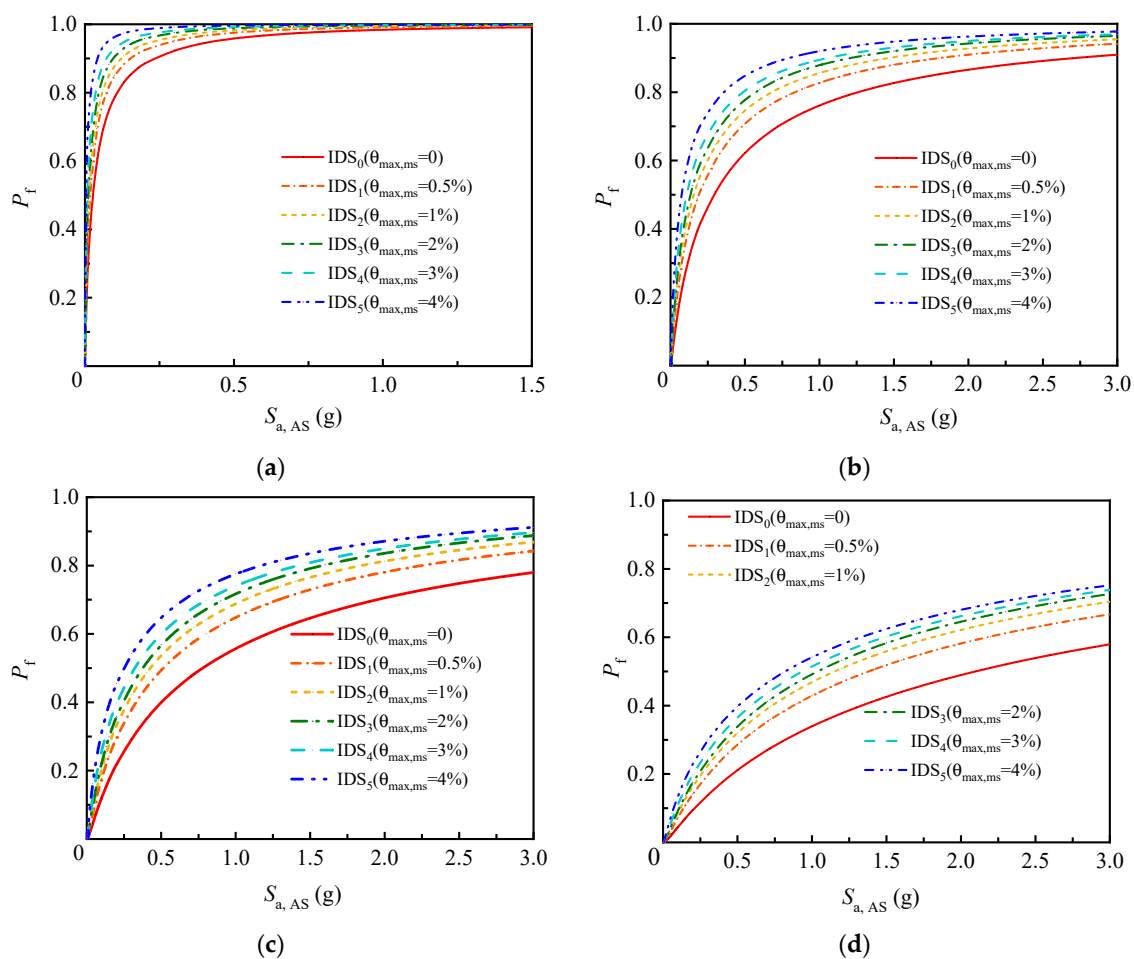


Figure 14. A comparison of fragility curves of low-ductility structures with different mainshock damage for aftershock records: (a) light limit state, (b) moderate limit state, (c) severe limit state, and (d) collapse (IDS₁-mainshock-induced damage state).

The results in Figure 14 demonstrate that the probability of the low-ductility structure in the IDS₁ state exceeding the light limit state increases by 4.6% compared with the structure in the IDS₀ state, at a spectral acceleration of 0.2 g, whereas the exceedance probability of the same structure in the IDS₅ state increases by 10.6% compared with that in the IDS₀ state. In the moderate limit state, when the spectral acceleration is 0.5 g, the exceedance probability of the low-ductility structure corresponding to the IDS₁ state increases by 9.1% compared with that in the IDS₀ state. Similarly, the exceedance probability of the structure in the IDS₅ state increases by 23.0% compared with that in the IDS₀ state, and the exceedance probability corresponding to the IDS₁ state increases by 14.1%. The

increment in the exceedance probability corresponding to different mainshock-induced damage states was calculated sequentially under the severe limit state and the collapse condition. Figure 15 presents the differences in failure probability between the intact low-ductility structure and various mainshock-induced damage states during aftershock motions, at each spectral acceleration. The results showed a substantial rise in the aftershock failure probability for mainshock-damaged structures across the four defined limit states: light, moderate, severe, and collapse. Notably, the low-ductility structure in the IDS_5 state consistently exhibited the highest increase in the probability of damage. For the light limit state, the failure probability of the structure with IDS_5 exhibited a maximum increase of 40.6% at $S_{a,AS} = 0.010$ g, where the failure probability exhibited maximum increments of 26.7%, 19.8%, 14.5%, and 9.6% at spectral accelerations of 0.015 g, 0.022 g, 0.024 g, and 0.025 g, respectively, for the structure with the other damage states (IDS_4 , IDS_3 , IDS_2 , and IDS_1); for the moderate limit state, where the failure probability exhibited increments of 29.5%, 21.9%, 17.7%, 13.9%, and 9.5% at spectral accelerations of 0.118 g, 0.156 g, 0.183 g, 0.183 g, and 0.183 g, respectively, for the structure with the damage states (IDS_5 , IDS_4 , IDS_3 , IDS_2 , IDS_1); and for the severe limit state, where the failure probability exhibited increments of 25.3%, 20.6%, 16.7%, 13.5%, and 9.6% at spectral accelerations of 0.352 g, 0.389 g, 0.520 g, 0.535 g, and 0.535 g, respectively, for the structure with the state (IDS_5 , IDS_4 , IDS_3 , IDS_2 , IDS_1). The exceedance probability of a low-ductility structure with the mainshock-induced damage is significantly different from that of an intact low-ductility structure at each limit state. Furthermore, as the mainshock-induced damage increases for low-ductility structures, the probability of exceeding various limit states under aftershocks also increases, although this increase is not substantial.

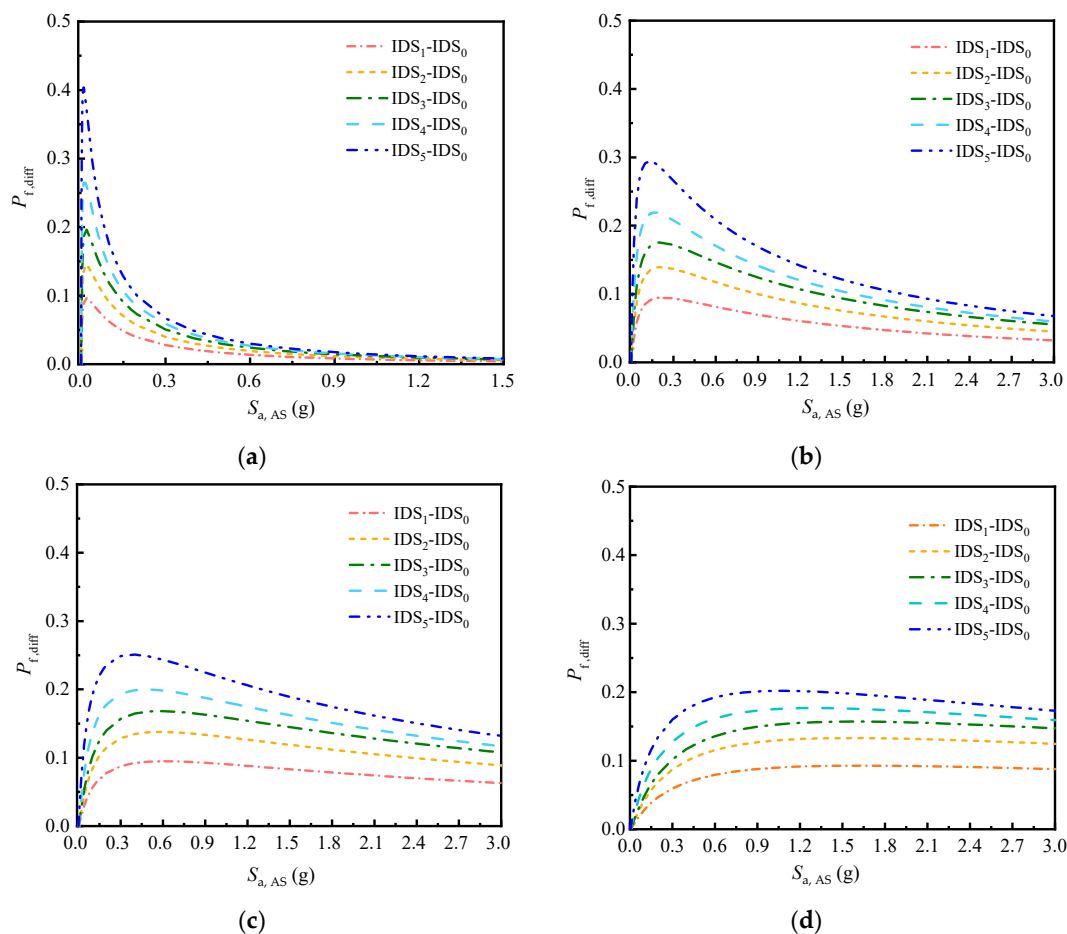


Figure 15. The difference of failure probability: (a) light limit state, (b) moderate limit state, (c) severe limit state, and (d) collapse (IDS_0 ($\theta_{max,ms} = 0$)) as a benchmark. Note: $P_{f,diff} = P_f (IDS_i) - P_f (IDS_0)$, IDS_i -mainshock-induced damage state, $i = 1 \dots 5$.

6. Conclusions

This study investigated the influence of different mainshock-induced damage states on the fragility of low-ductility RC structures during aftershocks by analyzing 65 actual aftershock ground motion records. The low-ductility structure was modeled via OpenSees, in which the flexure–shear interaction of RC and shear deformation in the joint zone was considered. In addition, the seismic fragility of low-ductility structures under aftershocks with various mainshock-induced damage states was established. This study provides a method for the post-earthquake assessment of earthquake-damaged structures to determine whether the structures can be directly restored for use during subsequent aftershocks. The main conclusions of this study are as follows:

- 1) The seismic demands of low-ductility structures with mainshock-induced damage during aftershocks are consistently higher than those of intact structures. Furthermore, the levels of mainshock-induced damage in low-ductility structures are directly correlated with the corresponding increase in seismic demand during the aftershocks.
- 2) The exceedance probability of low-ductility structures in the mainshock-induced damage states is higher than that of intact low-ductility structures in each limit state. Moreover, as the mainshock-induced damage increases, the probability of exceeding structural thresholds during aftershocks for the corresponding mainshock-damaged structure also increases.
- 3) Selecting the spectral acceleration corresponding to a 50% exceedance probability as an index to evaluate the collapse resistance capability of the structure, the results show that the collapse resistance capability of the structure decreases with the increase in mainshock-induced damage.
- 4) The exceedance probability of the limit states of low-ductility structures in the mainshock-induced damage state differs significantly from that of intact low-ductility structures. Specifically, when subjected to a ground motion with a spectral acceleration of 0.2 g, the exceedance probabilities corresponding to the IDS_1 state are 4.6% and 9.1% higher than that of the IDS_0 state for light and moderate damage states, respectively. Similarly, the exceedance probabilities of the IDS_5 state are 10.6% and 23.0% higher than that of the IDS_0 state for the same damage states, respectively.

Future works should employ multiple building models and multiple seismic motions to determine the initial damage and consider the effect of the uncertainties of the earthquake and the uncertainties of the structure.

Author Contributions: Conceptualization, B.Q. and J.H.; methodology, J.H., B.Q. and J.L.; software, J.L.; formal analysis, J.L.; data curation, B.Q.; writing—original draft, B.Q. and J.L.; writing—review and editing, J.H. All authors have read and agreed to the published version of the manuscript.

Funding: The support provided through the Key Research Development Program of Gansu Province (22YF7GA161) and the National Natural Science Foundation of China (51578273) is gratefully acknowledged.

Data Availability Statement: Data is contained within the article.

Conflicts of Interest: Author Jun Li was employed by the company Lanzhou Branch, Changzheng Engineering Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

References

1. Hauksson, E.; Jones, L.M.; Hutton, K. The 1994 Northridge earthquake sequence in California: Seismological and tectonic aspects. *J. Geophys. Res. Solid Earth* **1995**, *100*, 12335–12355. [\[CrossRef\]](#)
2. Huang, Y.; Wu, J.P.; Zhang, T.Z.; Zhang, D.N. Relocation of the M8.0 Wenchuan earthquake and its aftershock sequence. *Sci. China. Ser. D Earth Sci.* **2008**, *51*, 1703–1711. [\[CrossRef\]](#)
3. Shinohara, M.; Yamada, T.; Nakahigashi, K.; Sakai, S.; Mochizuki, K.; Uehira, K.; Ito, Y.; Azuma, R.; Kaiho, Y.; No, T.; et al. Aftershock observation of the 2011 off the Pacific coast of Tohoku Earthquake by using ocean bottom seismometer network. *Earth Planets Space* **2011**, *63*, 835–840. [\[CrossRef\]](#)

4. Fang, L.H.; Wu, J.P.; Wang, W.L.; Lü, Z.Y.; Wang, C.Z.; Yang, T.; Cai, Y. Relocation of the mainshock and aftershock sequences of Ms7.0 Sichuan Lushan earthquake. *Chin. Sci. Bull.* **2013**, *58*, 3451–3459. [\[CrossRef\]](#)
5. Paterson, E.; Re, D.; Wang, Z. *The 2008 Wenchuan Earthquake: Risk Management Lessons and Implications*; Risk Management Solutions, Inc.: Newark, CA, USA, 2008.
6. Lew, M.; Naeim, F.; Huang, S.C.; Lam, H.K.; Carpenter, L.D. Geotechnical and geological effects of the 21 September 1999 Chi-Chi earthquake, Taiwan. *Struct. Des. Tall Build.* **2000**, *9*, 89–106. [\[CrossRef\]](#)
7. Mahin, S.A. Effects of duration and aftershocks on inelastic design earthquakes. In Proceedings of the 7th World Conference on Earthquake Engineering, Tokyo, Japan, 8–13 September 1980.
8. Aschheim, M.; Black, E. Effects of prior earthquake damage on response of simple stiffness-degrading structures. *Earthq. Spectra* **1999**, *15*, 1–24. [\[CrossRef\]](#)
9. Sunasaka, Y.; Kiremidjian, A.S.; Toki, K. Strength demand spectra with uniform damage level in lifetime of structure. *J. Struct. Eng. A* **2002**, *48*, 523–530.
10. Amadio, C.; Fragiaco, M.; Rajgelj, S. The effects of repeated earthquake ground motions on the non-linear response of SDOF systems. *Earthq. Eng. Struct. Dyn.* **2003**, *32*, 291–308. [\[CrossRef\]](#)
11. Hatzigeorgiou, G.D.; Beskos, D.E. Inelastic displacement ratios for SDOF structures subjected to repeated earthquakes. *Eng. Struct.* **2009**, *31*, 2744–2755. [\[CrossRef\]](#)
12. Hatzigeorgiou, G.D. Behavior factors for nonlinear structures subjected to multiple near-fault earthquakes. *Comput. Struct.* **2010**, *88*, 309–321. [\[CrossRef\]](#)
13. Hatzigeorgiou, G.D.; Liolios, A.A. Nonlinear behaviour of RC frames under repeated strong ground motions. *Soil Dyn. Earthq. Eng.* **2010**, *30*, 1010–1025. [\[CrossRef\]](#)
14. Goda, K.; Salami, M.R. Inelastic seismic demand estimation of wood-frame houses subjected to mainshock-aftershock sequences. *B. Earthq. Eng.* **2014**, *12*, 855–874. [\[CrossRef\]](#)
15. Fragiaco, M.; Amadio, C.; Macorini, L. Seismic response of steel frames under repeated earthquake ground motions. *Eng. Struct.* **2004**, *26*, 2021. [\[CrossRef\]](#)
16. Tesfamariam, S.; Goda, K.; Mondal, G. Seismic vulnerability of reinforced concrete frame with unreinforced masonry infill due to main shock-aftershock earthquake sequences. *Earthq. Spectra* **2015**, *31*, 1427–1449. [\[CrossRef\]](#)
17. Han, R.L.; Li, Y.; van de Lindt, J.W. Assessment of Seismic Performance of Buildings with Incorporation of Aftershocks. *J. Perform. Constr. Fac.* **2015**, *29*, 04014088. [\[CrossRef\]](#)
18. Zhou, Z.; Xu, H.; Gardoni, P.; Lu, D.G.; Yu, X.H. Probabilistic demand models and fragilities for reinforced concrete frame structures subject to mainshock-aftershock sequences. *Eng. Struct.* **2021**, *245*, 112904. [\[CrossRef\]](#)
19. Zhou, Z.; Han, M.; Dong, Y.; Yu, X.H. Seismic resilience of corroded mid-rise reinforced concrete structures under mainshock-aftershock sequences. *Eng. Struct.* **2023**, *288*, 116192. [\[CrossRef\]](#)
20. Trapani, F.D.; Malavisi, M. Seismic fragility assessment of infilled frames subject to mainshock/aftershock sequences using a double incremental dynamic analysis approach. *Bull. Earthq. Eng.* **2019**, *17*, 211–235. [\[CrossRef\]](#)
21. Jalali, S.A.; Amini, A.; Mansouri, I.; Hu, J.W. Seismic collapse assessment of steel plate shear walls considering the mainshock-aftershock effects. *J. Constr. Steel Res.* **2021**, *182*, 106688. [\[CrossRef\]](#)
22. Nazari, N.; van de Lindt, J.W.; Li, Y. Effect of mainshock-aftershock sequences on woodframe building damage fragilities. *J. Perform. Constr. Fac.* **2015**, *29*, 04014036. [\[CrossRef\]](#)
23. Huang, L.J.; Zhou, Z.; Wei, Y.; Xie, Q.; Sun, X.Y. Seismic performance and resilience assessment of friction damped self-centering prestressed concrete frames. *Eng. Struct.* **2022**, *263*, 114346. [\[CrossRef\]](#)
24. Huang, L.J.; Clayton, P.M.; Zhou, Z. Seismic design and performance of self-centering precast concrete frames with variable friction dampers. *Eng. Struct.* **2021**, *245*, 112863. [\[CrossRef\]](#)
25. Huang, L.J.; Zeng, B.; Zhou, Z.; Zhang, W.Q.; Wei, Y.; Li, C.Y. Seismic behavior and reliability of variable friction damped self-centering prestressed concrete frames considering bolt bearing. *Soil. Dyn. Earth. Eng.* **2023**, *164*, 107643. [\[CrossRef\]](#)
26. Huang, L.J.; Wen, H.W.; Jiang, K.X.; Wei, Y.; Zhou, Z.; Xie, Q.; Qian, Z.D. Seismic performance of partial self-centering prestressed concrete frames with friction dampers. *Structures* **2024**, *62*, 106268. [\[CrossRef\]](#)
27. D’Aragona, M.G.; Polese, M.; Elwood, K.J.; Shoraka, M.B.; Prota, A. Aftershock collapse fragility curves for non-ductile RC buildings: A scenario-based assessment. *Earthq. Eng. Struct. Dyn.* **2017**, *46*, 2083–2102. [\[CrossRef\]](#)
28. Jeon, J.S.; Lowes, L.N.; DesRoches, R.; Brilakis, I. Fragility curves for non-ductile reinforced concrete frames that exhibit different component response mechanisms. *Eng. Struct.* **2015**, *85*, 127–143. [\[CrossRef\]](#)
29. Sarno, L.D.; Pugliese, F. Effects of mainshock-aftershock sequences on fragility analysis of RC buildings with ageing. *Eng. Struct.* **2021**, *232*, 111837. [\[CrossRef\]](#)
30. Han, R.; Li, Y.; van de Lindt, J. Impact of aftershocks and uncertainties on the seismic evaluation of non-ductile reinforced concrete frame buildings. *Eng. Struct.* **2015**, *100*, 149–163. [\[CrossRef\]](#)
31. Elwood, K.J.; Matamoros, A.B.; Wallace, J.W.; Lehman, D.E.; Heintz, J.A.; Mitchell, A.D.; Moore, M.A.; Valley, M.T.; Lowes, L.N.; Comartin, C.D.; et al. Update to ASCE/SEI 41 concrete provisions. *Earthq. Spectra* **2007**, *23*, 493–523. [\[CrossRef\]](#)
32. Zhu, L.; Elwood, K.J.; Haukaas, T. Classification and seismic safety evaluation of existing reinforced concrete columns. *ASCE J. Struct. Eng.* **2007**, *133*, 1316–1330. [\[CrossRef\]](#)
33. Elwood, K.J. Modelling failures in existing reinforced concrete columns. *Can. J. Civ. Eng.* **2004**, *31*, 846–859. [\[CrossRef\]](#)

34. Miano, A.; Jalayer, F.; Ebrahimian, H.; Prota, A. Cloud to IDA: Efficient fragility assessment with limited scaling. *Earthq. Eng. Struct. Dyn.* **2018**, *47*, 1124–1147. [CrossRef]
35. Wen, Y.K.; Ellingwood, B.; Bracci, J.M. *Vulnerability Function Framework for Consequence-Based Engineering*; MAE Center Project DS-4 Report 04-04; 2004. Available online: <https://www.ideals.illinois.edu/items/9241> (accessed on 10 May 2024).
36. Lowes, L.N.; Altoontash, A. Modeling reinforced-concrete beam-column joints subjected to cyclic loading. *ASCE J. Struct. Eng.* **2003**, *129*, 1686–1697. [CrossRef]
37. Song, M.C. Study on Modeling Method of Reinforced Concrete Beam-Column Joint Core. Master's Thesis, Chongqing University, Chongqing, China, 2009. (In Chinese).
38. Mitra, N.; Lowes, L.N. Evaluation, calibration, and verification of a reinforced concrete beam-column joint model. *ASCE J. Struct. Eng.* **2007**, *133*, 105–120. [CrossRef]
39. Paspuleti, C. Seismic Analysis of an Older Reinforced Concrete Frame Structure. Master's Thesis, University of Washington, Seattle, WA, USA, 2002.
40. McKenna, F. OpenSees: A framework for earthquake engineering simulation. *Comput. Sci. Eng.* **2011**, *13*, 58–66. [CrossRef]
41. Zhao, J.; Sritharan, S. Modeling of strain penetration effects in fiber-based analysis of reinforced concrete structures. *ACI Struct. J.* **2007**, *104*, 133–141.
42. Ancheta, T.D.; Darragh, R.B.; Stewart, J.P.; Seyhan, E.; Silva, W.J.; Chiou, B.S.-J.; Wooddell, K.E.; Graves, R.W.; Kottke, A.R.; Boore, D.M.; et al. NGA-West2 database. *Earthq. Spectra.* **2014**, *30*, 989–1005. [CrossRef]
43. Goda, K.; Taylor, C.A. Effects of aftershocks on peak ductility demand due to strong ground motion records from shallow crustal earthquakes. *Earthq. Eng. Struct. Dyn.* **2012**, *41*, 2311–2330. [CrossRef]
44. Raghunandan, M.; Liel, A.B.; Luco, N. Aftershock collapse vulnerability assessment of reinforced concrete frame structures. *Earthq. Eng. Struct. Dyn.* **2014**, *44*, 419–439. [CrossRef]
45. Xu, J.F.; Chen, J.; Ding, G. Fragility analysis and life cycle cost assessment of RC frame under mainshock-aftershock seismic sequences through IDA. *Earthq. Eng. Eng. Dyn.* **2015**, *35*, 206–212. (In Chinese)
46. Yu, X.H.; Lu, D.G.; Li, B. Estimating uncertainty in limit state capacities for reinforced concrete frame structures through pushover analysis. *Earthq. Struct.* **2016**, *10*, 141–161. [CrossRef]

Disclaimer/Publisher's Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.