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Predictive Equations for Modeling Shear Behavior in RC Columns with Non-Seismic Details for Enhanced Seismic Performance Assessment

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Abstract: Accurate modeling of the hysteretic behavior of reinforced concrete columns with non-seismic detailing is crucial for assessing the seismic performance of reinforced concrete structures. While shear spring models are commonly used to capture nonlinear shear deformation in shear-critical reinforced concrete columns, determining appropriate parameters for these models remains challenging and often relies on expert judgment. This study proposes predictive equations for shear spring parameters that can be integrated into fiber-section elements to enhance the accuracy of modeling reinforced concrete columns. Specifically, empirical equations are developed for the nonlinear modeling parameters of the Pinching4 model implemented in OpenSees. Experimental data of flexural–shear and shear-critical reinforced concrete columns are used to calibrate the modeling parameters for predicting strength, stiffness, pinching, and cyclic degradation behavior. The predictive equations, derived through regression analysis, are validated against experimental data and demonstrated relatively high accuracy in capturing the shear behavior. To evaluate the impact of the shear behavior of reinforced concrete columns on the seismic fragility of non-conforming reinforced concrete frames, two types of numerical models are established with and without the Pinching4 material using the proposed predictive equations. Analytical results indicate that neglecting the appropriate shear behavior of reinforced concrete columns could lead to an overestimation of the seismic performance of non-conforming reinforced concrete frames.



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1. Introduction

Seismic design standards have been continuously improved to mitigate potential earthquake damage to buildings in seismically active regions. Despite these efforts, a significant number of reinforced concrete (RC) structures, particularly those constructed before the implementation of modern seismic codes, remain vulnerable due to inadequate seismic design and detailing. In South Korea, for instance, seismic design codes were first mandated for new buildings in 1988 and have since undergone continuous revisions and expansions [1]. Consequently, RC members in older buildings often exhibit non-conforming cross-sectional details, such as widely spaced transverse reinforcements, the absence of seismic hooks, and insufficient anchorage length of stirrups [2–4]. These deficiencies can significantly compromise the ductility of RC members under seismic loads, increasing their susceptibility to brittle failure modes such as reinforcing bar rupture and shear failure [5].

Past earthquake damage consistently underscores the risks associated with such structures. To enhance the seismic performance of existing buildings and improve future designs, significant research efforts have been dedicated to developing effective retrofitting strategies and refining seismic design methodologies [6–9]. Successful mitigation of earthquake damage necessitates a comprehensive understanding of the seismic behavior of such structures. This requires an accurate assessment of seismic demands and the identification of critical members that are most susceptible to damage through reliable analysis. However, accurately simulating the nonlinear shear behavior of RC members in analysis programs remains a challenge, which could significantly affect the accuracy of structural performance evaluations such as seismic fragility analysis.

Seismic fragility analysis is a methodology used to evaluate a structure's vulnerability to specific ground motion levels by assessing the likelihood of the structure surpassing predefined damage thresholds [10–12]. The concept of seismic fragility functions was initially introduced in the 1970s for nuclear power plant containment structures and was further refined by Kendy and Ravindra (1984) [11]. Subsequent research has extended this concept to various structures, including buildings and bridges [10,12]. Seismic fragility functions serve as performance evaluation tools, quantifying the probability of specific damage states occurring in a target structure based on various ground motion parameters like peak ground acceleration (PGA), spectral acceleration (SA), and others. By evaluating a structure's seismic performance considering diverse ground motion parameters, it becomes possible to rapidly anticipate potential damage to the structure during random seismic events.

In general, seismic fragility functions provide a quantitative assessment of the response to ground motion using damage probability, offering predictions as they leverage nonlinear numerical models of the target structure. However, it is crucial to note that the probability of exceeding a damaged state relies on the outcomes of time history analysis with detailed nonlinear numerical models, which can be computationally expensive and challenging to generate with high fidelity. These limitations are particularly acute for RC structures with non-conforming details, where shear behavior plays a predominant role.

To accurately evaluate the response of such structures, a reliable nonlinear numerical model for members exhibiting predominantly shear-critical behavior is crucial. While existing nonlinear analysis programs often face challenges in modeling this behavior, extensive research has been dedicated to developing shear spring models applicable to fiber-section and finite-length models [13–20]. Elwood (2004) [13] proposed a model using a limit state material model to simulate the behavior of shear-critical RC columns. This approach combines fiber-section elements for flexural behavior, elastic rotational spring elements to represent bar-slip behavior, and translational spring based on the limit state material model to capture shear behavior. The degradation in axial capacity due to damage from shear behavior is simulated by applying the limit state material as an axial spring. Haselton et al. (2016) [21] presented a concentrated plasticity model utilizing the modified Ibarra–Medina–Krawinkler deterioration model [22] to represent member behavior through a rotational spring. Leborgne and Ghannoum (2014) [15] proposed a model to capture the initial lateral strength degradation using rotation of the plastic hinge region at the column end. The flexural behavior of the column can be simulated using a fiber-section-based element model capable of representing both strength damage and stiffness damage.

Jeon et al. (2015) [17] investigated the seismic fragility of RC frames, considering shear deformation in beam–column joints. Five different joint models were studied: rigid joint model, joint shear response without anchorage failure, joint shear response with anchorage failure, column shear response, and joint and column shear response. The rotational deformation of the joint panel zone was modeled using the Pinching4 material model [23] as a

rotational spring. The shear behavior in the column was captured by the Elwood (2004) [13] model, implemented as a translational limit state model. To account for bar slip, a zero-length fiber-section element was employed in the model, effectively capturing the bond-slip behavior between reinforcing bars and concrete. Huang and Huang (2021) [18] conducted a seismic vulnerability assessment of RC bridges considering the influence of chloride-induced corrosion of shear-critical RC piers. The flexural behavior of the RC piers was modeled using a force-based beam-column element [24], which is based on the iterative force-based formulation. The bond-slip effect of the longitudinal rebar was represented using a rotational zero-length spring element. The behavior for three failure modes (flexural, flexural-shear, and shear failure) was implemented using translational spring elements, modifying the Elwood (2004) [13] model of the limit state material model [19,20].

The aforementioned models aim to capture complex behaviors in RC members under cyclic loading, including strength and stiffness degradation, pinching effects, and other phenomena that are challenging to represent in traditional models. Effective implementation of shear springs relies on accurate parameters tailored to each RC member's characteristics. While many researchers have proposed equations to predict these parameters based on the properties of RC columns, limitations exist. These limitations encompass the inability to account for pinching effects [13,14], difficulty in modeling stiffness or strength degradation due to cyclic loading [14,15], and predictions that are often based on experimental data from flexural failure mode [14]. Consequently, these limitations can significantly hinder the accurate prediction of member behavior under seismic loads, especially for non-conforming RC columns.

Hence, to address these challenges, this study proposes predictive equations for shear spring parameters to enhance the simulation accuracy of capacity degradation and pinching effect simulations in RC columns subjected to cyclic loading. Regression equations utilizing fundamental physical properties are provided to streamline parameter derivation for the Pinching4 [23] model within OpenSees [25], thereby reducing computational effort compared to existing methods. The effectiveness of the proposed predictive equations is validated through two numerical models: one incorporating the new shear spring parameters and another baseline model without them. These models are employed to evaluate the impact of shear deformation in vertical members on the seismic fragility of a non-conforming RC frame.

2. Shear Spring Model

2.1. Analytical Model Types

The nonlinear numerical models for RC members can be broadly classified into two main categories, as illustrated in Figure 1 [26,27]. The concentrated plasticity model simplifies the nonlinear behavior by lumping it at specific locations like springs or hinges. These hinges are calibrated based on the nonlinear moment-curvature relationship, typically derived from cross-sectional analysis or experimental results of RC members. In contrast, the distributed plasticity model offers a more detailed representation by spreading the nonlinearity throughout the member. This category further branches into three sub-types: finite-length hinge models, fiber-section models, and finite element models [27].

The fiber-section model depicts the cross-section of the RC member using a fiber model with uniaxial material properties. Unlike the concentrated plasticity spring model, it does not necessitate the calibration of moment-curvature relationships, and it can promptly calculate the response based on axial force and moment. The finite-length model applies to a nonlinear model in specific regions, such as plastic hinge lengths, while retaining elastic behavior elsewhere. The most sophisticated approach is the finite element model. This

model can systematically capture shear deformation and failure but requires significant computational resources.

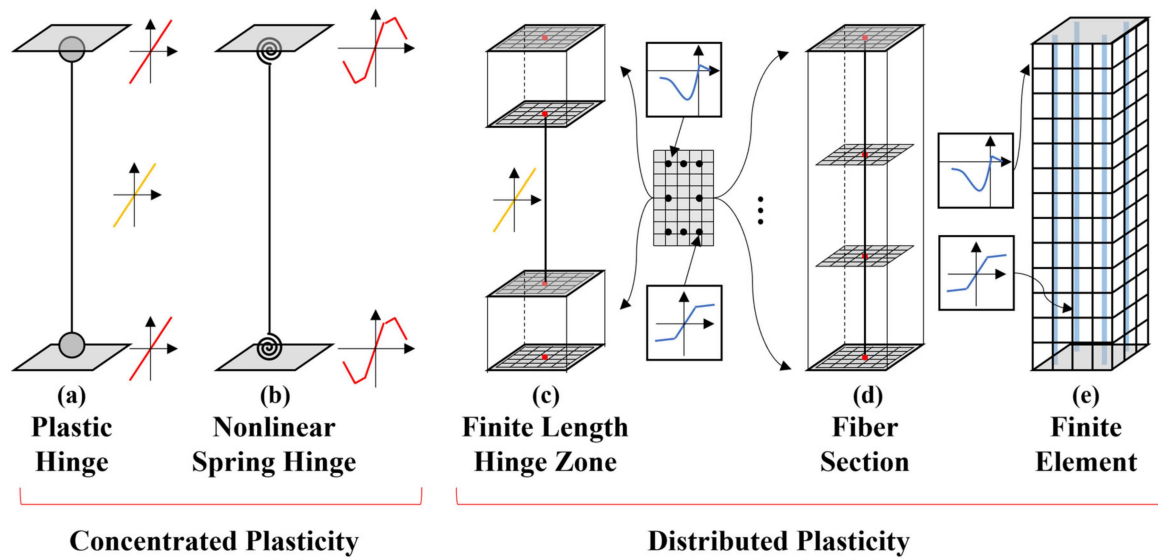


Figure 1. Idealized modeling approaches for beam–column elements (adapted from Deierlein et al., 2010) [27]: (a) rigid-plastic hinge model; (b) inelastic spring model with hysteretic properties; (c) finite-length hinge zone at member ends; (d) fiber-section model along the member length; and (e) finite element model.

While the fiber-section model is effective for capturing flexural behavior, it falls short in representing the shear-critical behavior of RC members, especially in cases where shear deformation significantly influences the response. Therefore, this study employs a model that incorporates a translational shear spring into the fiber-section-based element. The details of the model are presented in Section 3.

2.2. Pinching4 Model

The structural analysis program employed for simulating the shear spring in this study is OpenSees [25], and the shear spring element is modeled using a zero-length element with the Pinching4 material [23]. This study aims to develop predictive equations for the shear spring parameters in the Pinching4 model. Thus, this section provides a brief overview of the parameters required for the Pinching4 model, as detailed in Lowes et al., 2003 [23].

As depicted in Figure 2, the Pinching4 model requires parameters for constructing a four-point backbone curve, typically determined by idealizing pushover test results. The four points are defined as the cracking point (D_{cr} , F_{cr}), yield point (D_{yi} , F_{yi}), maximum force point (D_{mx} , F_{mx}), and ultimate point (D_{ul} , F_{ul}), where D and F denote displacement and force, respectively, at each state. Additionally, pinching behavior is modeled using three parameters, κ_{rd} , κ_{rf} , and κ_{uf} , characterizing reloading displacement, reloading force, and unloading force, respectively. These parameters are defined as the ratio of the force or displacement at the pinching point to the maximum force or displacement of the current cycle, as shown in Figure 2a. Stiffness and strength degradation, indicative of reduced member performance under cyclic loading, are captured by three parameters (k_i , $(d_{max})_i$, and $(f_{max})_i$), as illustrated in Figure 2b–d. Here, i denotes the current displacement increment, and thus k_i , $(d_{max})_i$, and $(f_{max})_i$ represent the current unloading stiffness, deformation, and envelope maximum strength, respectively. The parameter $(d_{max})_i$ defines the end of the reload cycle for increasing deformation demand. These parameters are calculated using Equations (1)–(3).

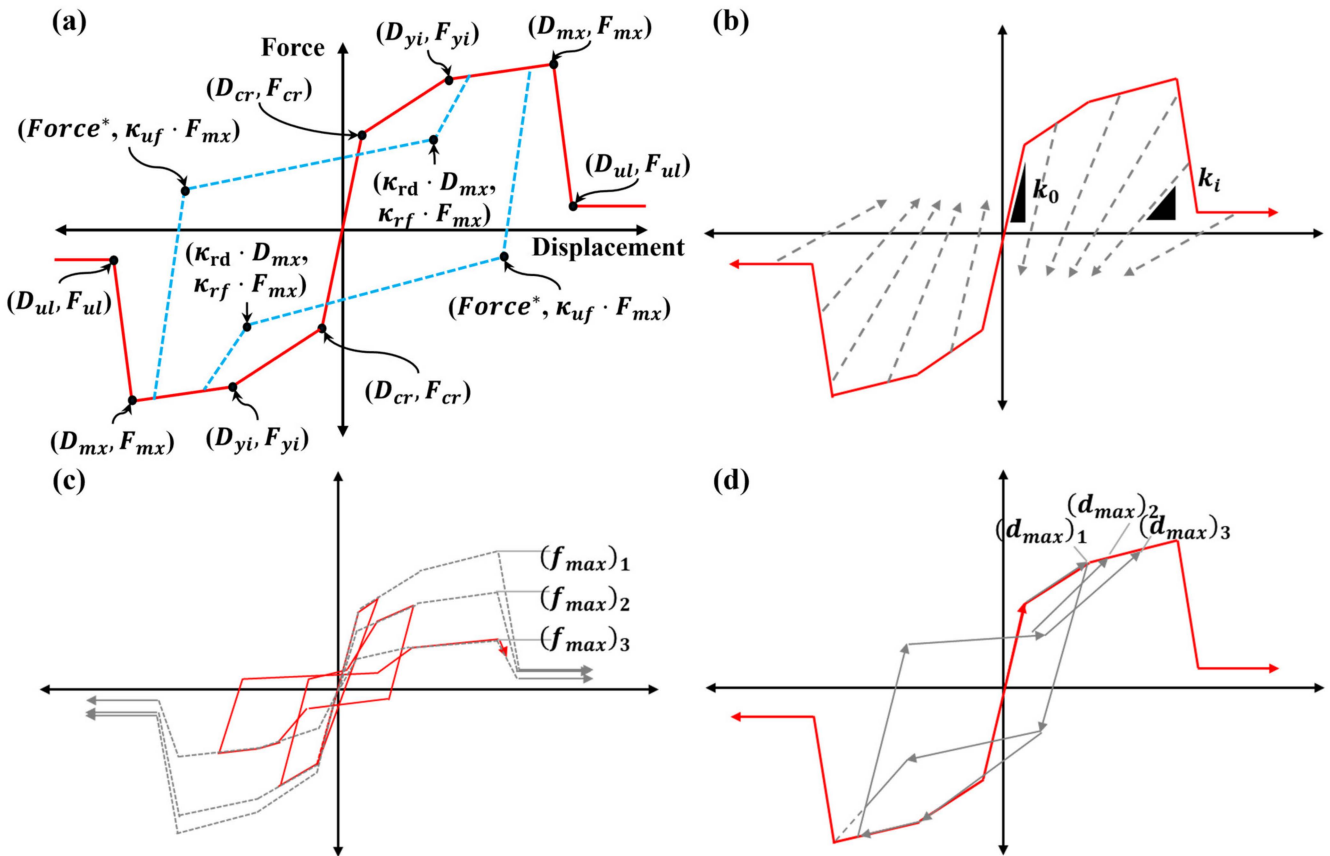


Figure 2. Pinching4 material model parameters (adapted from Lowes et al., 2003) [23]: (a) backbone curve and pinching; (b) unloading stiffness degradation; (c) strength degradation; (d) and reloading stiffness degradation.

$$k_i = k_0 \times (1 - \delta k_i) \quad (1)$$

$$(d_{max})_i = (d_{max})_0 \times (1 + \delta d_i) \quad (2)$$

$$(f_{max})_i = (f_{max})_0 \times (1 - \delta f_i) \quad (3)$$

Here, k_0 is the initial unloading stiffness and $(d_{max})_0$ is the maximum historic deformation demand. $(f_{max})_0$ is the initial envelope maximum strength for the case of no damage. δk_i , δd_i and δf_i are the stiffness damage value, the reloading stiffness damage value, and the strength damage value at t_i , respectively. The parameters δk_i , δd_i , and δf_i are defined using Equations (4)–(6).

$$\delta k_i = \left(gK1 \cdot (\tilde{d}_{max})^{gK2} + gK2 \cdot \left(\frac{E_i}{E_{mono}} \right)^{gK4} \right) \leq gKLim \quad (4)$$

$$\delta d_i = \left(gD1 \cdot (\tilde{d}_{max})^{gD2} + gD2 \cdot \left(\frac{E_i}{E_{mono}} \right)^{gD4} \right) \leq gDLim \quad (5)$$

$$\delta f_i = \left(gF1 \cdot (\tilde{d}_{max})^{gF2} + gF2 \cdot \left(\frac{E_i}{E_{mono}} \right)^{gF4} \right) \leq gFLim \quad (6)$$

Here, $\tilde{d}_{max}$ is the ratio of maximum to minimum historic displacement demands defining failure. E_i represents the accumulated hysteretic energy, and E_{mono} is the energy required to achieve the deformation that defines failure. The parameter used with $(\tilde{d}_{max})$

in Equations (4)–(6) is a damage parameter related to the displacement that occurred during cyclic loading, and the parameter used with E_i/E_{mono} is related to the amount of accumulated energy. The parameters $gN1$ to $gN4$ and $gNLim$ (where N is K , D , or F) used in Equations (4)–(6) are damage parameters input by the user. Among them, $gN1$ and $gN3$ are displacement-based damage parameters, $gN2$ and $gN4$ are accumulated energy-based damage parameters, and $gNLim$ defines the maximum value of the damage index. The parameters $\tilde{d}_{max}$ and E_i are defined as in Equations (7) and (8), respectively.

$$\tilde{d}_{max} = \text{maximum} \left[\frac{(d_{max})_i}{def_{max}}, \frac{(d_{min})_i}{def_{min}} \right] \quad (7)$$

$$E_i = \int_{\text{load history}} dE \quad (8)$$

Here, def_{max} and def_{min} represent positive and negative deformations defining failure. $(d_{max})_i$ and $(d_{min})_i$ are, respectively, the maximum and minimum historic deformation demands.

Utilizing all 12 damage parameters outlined in Equations (4)–(6) can lead to considerable computational effort to identify optimal parameters for the model. To address this challenge, Lee and Han (2018) [16] introduced a simplified approach based on the damage model presented in Lignos and Krawinkler (2011) [28] and Haselton et al. (2016) [21]. This approach focuses solely on energy-based damage parameters shown in Equations (9)–(11), excluding displacement-based ones. A comparison between damage calculated using the comprehensive model (Equations (4)–(6)) and the simplified model (Equations (9)–(11)) demonstrated good agreement, with a median of 0.98 and a standard deviation of 0.04 [16]. Therefore, to enhance computational efficiency without sacrificing accuracy, this study adopts the energy-based damage parameters presented in Equations (9)–(11).

$$\delta k_i = gK2 \cdot \left(\frac{E_i}{E_{mono}} \right) \leq gKLim \quad (9)$$

$$\delta d_i = gD2 \cdot \left(\frac{E_i}{E_{mono}} \right) \leq gDLim \quad (10)$$

$$\delta f_i = gF2 \cdot \left(\frac{E_i}{E_{mono}} \right) \leq gFLim \quad (11)$$

3. Predictive Equations for Shear Spring Parameters

3.1. Selected Experimental Data

To develop predictive equations for shear spring parameters, this study employed a manual calibration process using experimental data from previous research and specimen properties. The experimental dataset was obtained from the Structural Performance Database in the Pacific Earthquake Engineering Research (PEER) Center [29]. A stringent filtering process based on eight criteria was implemented to select relevant data:

1. Rectangular cross-section;
2. Reverse cyclic loading;
3. Concrete compressive strength less than 35 MPa;
4. Shear-span-to-depth ratio less than 3.5;
5. Transverse reinforcement ratio less than 0.0025;
6. Section aspect ratio (height/width) lower than 2.0;
7. Confinement types limited to rectangular ties (Rs), rectangular and diagonal ties (RDs), and rectangular ties with J-hooks (RJs);
8. Single specimen tested (cantilever (C) or double curvature (DC)).

Table A1 in the Appendix A summarizes the properties of the 40 columns selected from a total of 256 rectangular RC columns in the PEER database (Appendix A). It should be noted that specimens tested with a double-ended configuration were excluded due to potential interaction between the connected specimens. As shown in Table A1, the column failure mode is classified as flexure-critical (F), flexure–shear-critical (FS), or shear-critical (S). This rigorous data selection and manual calibration process using 40 datasets is expected to yield accurate and relevant shear spring parameters for the chosen model, thereby enhancing the reliability of subsequent numerical simulations.

3.2. Calibration of Model Parameters

In this study, the Pinching4 material model was employed to simulate the shear behavior of the member as a transverse shear spring, as depicted in Figure 3. Rotational bar-slip behavior was captured using a zero-length element with a fiber section in terms of the rotational degree of freedom. The column member itself was represented as a displacement-based element to capture its flexural behavior. Model parameters were calibrated through an iterative process, incorporating the following assumptions:

1. Maximum energy dissipation capacity coefficient: This is defined as the ratio of the total area under the backbone curve to the area up to the yield point.
2. Residual strength and displacement: Residual strength (F_{ul}) is assumed to be zero for simplicity, and the corresponding displacement (D_{ul}) is taken as 1.5 times the maximum specimen displacement.

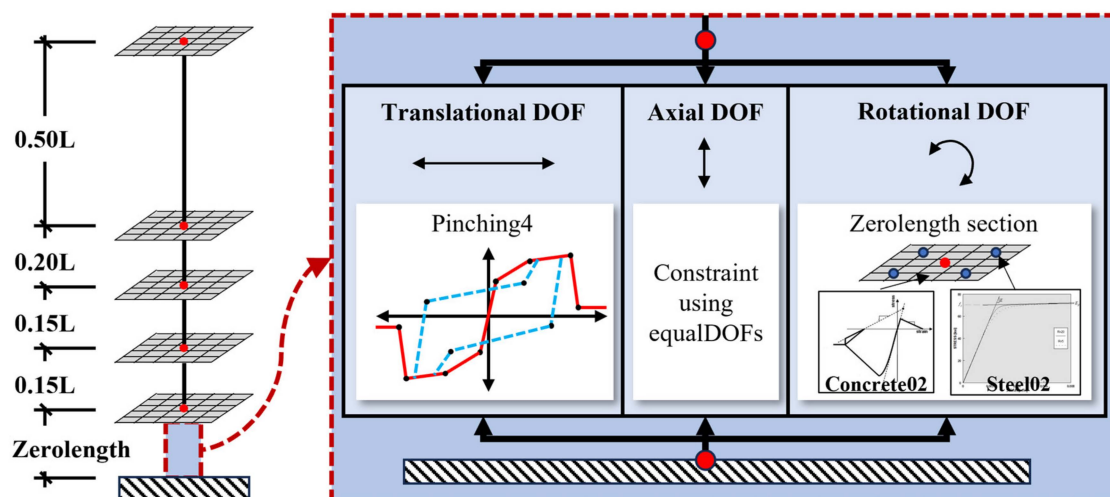


Figure 3. Description of proposed column model.

The Pinching4 model assumes structural failure when the current cumulative dissipated energy surpasses the maximum energy dissipation capacity. Therefore, the second assumption ensures this limit is not reached prematurely, enabling an accurate representation of the hysteretic behavior up to specimen failure.

The calibration examples for RC columns, as presented in Figure 4, demonstrate good agreement between the simulated response and experimental data. However, Figure 5a–c reveal cases where convergence issues arise for members with a very small shear-span-to-depth ratio (≤ 1.0). Furthermore, as illustrated in Figure 5e,f, adjustments to shear spring parameters have minimal influence on the analysis results for specimens exhibiting a flexural failure mode. This can be attributed to the shear spring strength exceeding that of the fiber-section-based element, leading to predominantly flexural behavior. Including the calibrated parameters for these cases in the regression analysis could potentially compromise the reliability of the resulting model. As a result, 14 specimens exhibiting these

characteristics were excluded from the regression analysis. The calibrated parameters for the remaining columns are presented in Table A2 in the Appendix A.

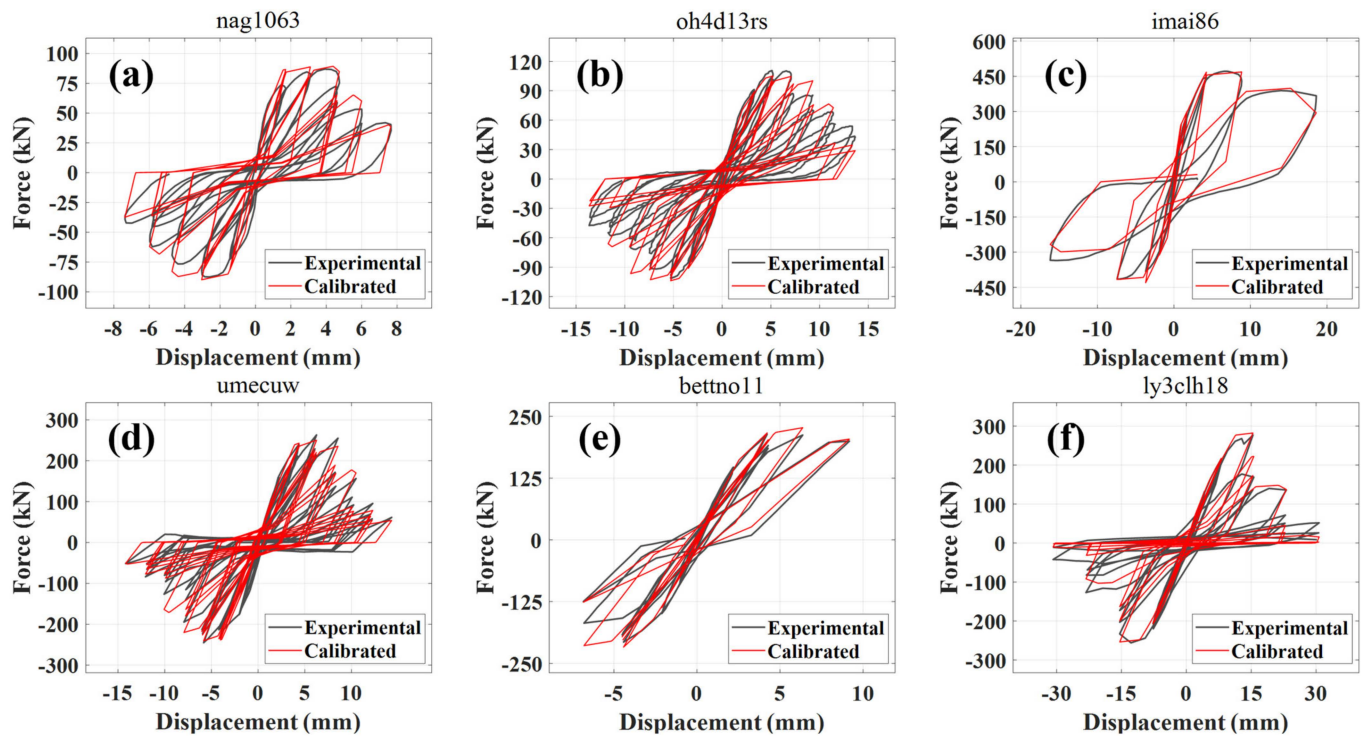


Figure 4. Comparison of experimental results (gray solid line) and calibrated model predictions (red solid line) for well-matched cases: (a) nag1063; (b) oh4d13rs; (c) imai86; (d) umecuw; (e) bettno11; and (f) ly3clh18.

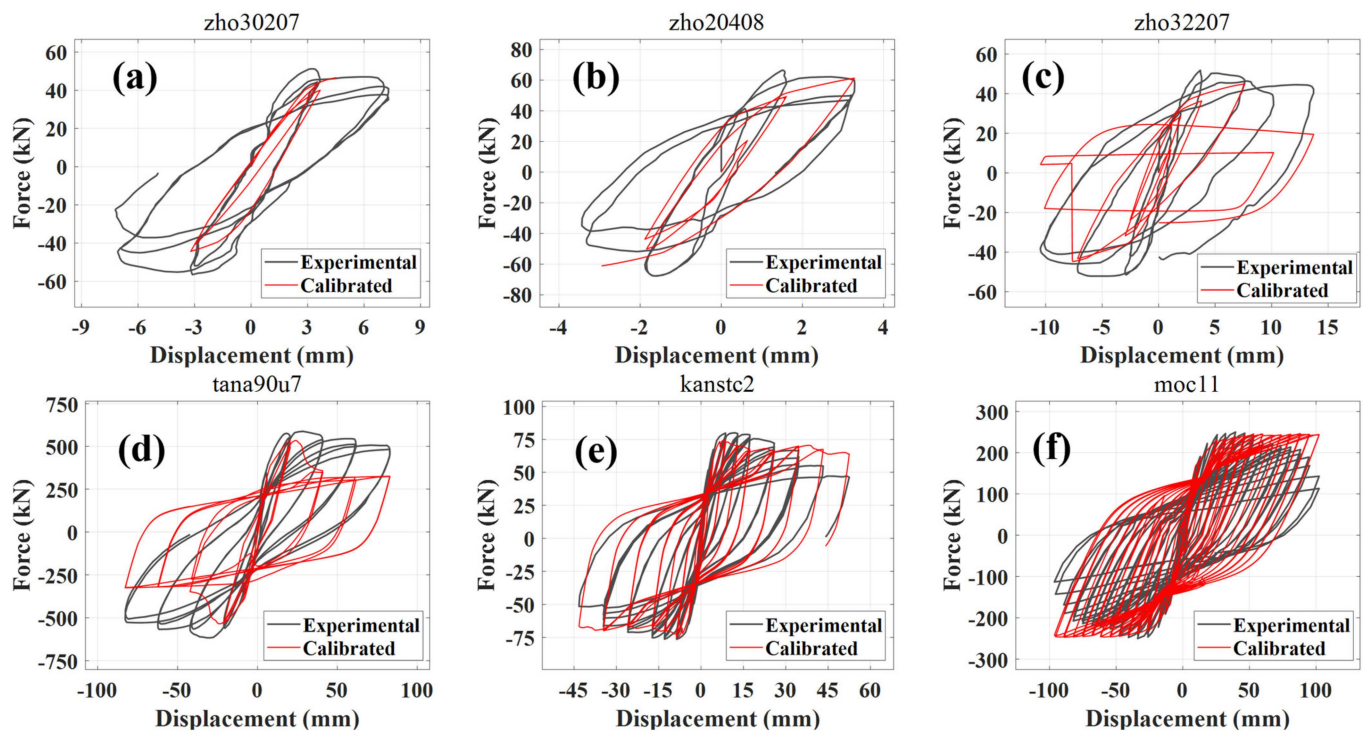


Figure 5. Comparison of experimental results (gray solid line) and calibrated model predictions (red solid line) for cases exhibiting discrepancies: (a) zho20408; (b) zho30207; (c) zho32207; (d) tana90u7; (e) kanstc2; and (f) moc11.

3.3. Development of Predictive Equations

A regression analysis was conducted to establish prediction equations based on specimen properties, utilizing the calibrated shear spring parameters from Section 3.2. The considered properties include column section width (b), section height (h), column length (L), transverse reinforcement spacing (s), clear cover (cc), gross sectional area (A_g), concrete compressive strength (f'_c), yield strengths of longitudinal and transverse rebars (f_{yl} , f_{yt}), a confinement factor (κ_c) calculated by the Cusson–Paultre model [30], longitudinal and transverse reinforcement ratios (ρ_l , ρ_t), and the ratio of applied axial load to the axial compressive capacity of the column (ρ_{Pn}).

Linear regression analysis with exponential and natural logarithmic functions was employed to develop prediction equations for shear spring parameters. Regression analysis for backbone curve parameters was performed exclusively for three points (crack, yield, and maximum). For the residual strength point, the force was assumed to be zero, and the displacement was assumed to be 1.5 times the maximum point displacement; thus, a separate regression analysis was not conducted for this point. The pinching parameter, potentially ranging from -1 to 1 , was adjusted by adding 1 to accommodate potential negative values. Similarly, the degradation parameter was adjusted by adding 1 to avoid zero values. The proposed prediction equations for each parameter are shown in Equations (12)–(23).

Figure 6 presents the regression analysis results of the proposed prediction equations for shear spring parameters. The coefficient of determination (R^2) for the backbone curve parameters ranges from 0.8078 to 0.9522 , indicating a generally good predictive capability. However, the accuracy for predicting F_{cr} and D_{cr} is comparatively lower.

$$D_{cr} = e^{-1.82} \cdot b^{-1.69} \cdot h^{-1.89} \cdot s^{0.93} \cdot L^{1.83} \cdot f'_c{}^{0.02} \cdot f_{yl}{}^{0.69} \cdot f_{yt}{}^{1.48} \cdot \rho_l{}^{0.33} \cdot \rho_t{}^{1.06} \cdot \rho_{Pn}{}^{-0.33} \cdot \kappa_c^{-3.58} \quad (12)$$

$$D_{yi} = e^{-7.25} \cdot b^{-0.98} \cdot h^{-0.89} \cdot L^{1.90} \cdot f'_c{}^{0.28} \cdot f_{yl}{}^{1.77} \cdot f_{yt}{}^{0.52} \cdot \rho_l{}^{0.47} \cdot \rho_t{}^{-0.12} \cdot \rho_{Pn}{}^{-1.99} \cdot \kappa_c^{-1.99} \quad (13)$$

$$D_{mx} = e^{-8.25} \cdot b^{-0.55} \cdot h^{-0.90} \cdot L^{1.88} \cdot f'_c{}^{1.22} \cdot f_{yl}{}^{0.36} \cdot f_{yt}{}^{0.15} \cdot \rho_l{}^{0.07} \cdot \rho_t{}^{-0.53} \cdot \rho_{Pn}{}^{3.10} \cdot \kappa_c^{3.10} \quad (14)$$

$$F_{cr} = e^{2.82} \cdot b^{-0.16} \cdot h^{1.43} \cdot f'_c{}^{-0.20} \cdot \rho_l{}^{0.47} \cdot \rho_t{}^{0.43} \cdot \rho_{Pn}{}^{0.19} \cdot \kappa_c^{-2.54} \quad (15)$$

$$F_{yi} = e^{1.06} \cdot b^{0.42} \cdot h^{0.99} \cdot f'_c{}^{-0.05} \cdot \rho_l{}^{0.59} \cdot \rho_t{}^{0.22} \cdot \rho_{Pn}{}^{0.12} \cdot \kappa_c^{-0.87} \quad (16)$$

$$F_{mx} = e^{-0.47} \cdot b^{0.35} \cdot h^{1.03} \cdot f'_c{}^{0.28} \cdot \rho_l{}^{0.46} \cdot \rho_t{}^{0.17} \cdot \rho_{Pn}{}^{0.14} \cdot \kappa_c^{-0.68} \quad (17)$$

$$\kappa_{rd} = e^{1.98} \cdot A_g^{-0.12} \cdot s^{0.15} \cdot cc^{0.16} \cdot f'_c{}^{0.19} \cdot f_{yl}{}^{-0.35} \cdot f_{yt}{}^{-0.03} \cdot \rho_l{}^{-0.15} \cdot \rho_t{}^{0.13} \cdot \rho_{Pn}{}^{-0.03} - 1 \quad (18)$$

$$\kappa_{rf} = e^{-1.66} \cdot A_g^{0.13} \cdot f'_c{}^{0.48} \cdot f_{yl}{}^{-0.35} \cdot f_{yt}{}^{0.01} \cdot \rho_l{}^{-0.19} \cdot \rho_t{}^{-0.02} \cdot \rho_{Pn}{}^{0.04} \cdot \kappa_c^{0.86} - 1 \quad (19)$$

$$\kappa_{uf} = e^{-1.95} \cdot A_g^{-0.05} \cdot f'_c{}^{-0.50} \cdot f_{yl}{}^{-0.01} \cdot f_{yt}{}^{1.03} \cdot \rho_l{}^{0.10} \cdot \rho_t{}^{0.38} \cdot \rho_{Pn}{}^{-0.36} \cdot \kappa_c^{-2.42} - 1 \quad (20)$$

$$\alpha_{rk} = e^{-1.73} \cdot A_g^{0.58} \cdot L^{-0.71} \cdot f'_c{}^{-1.04} \cdot f_{yl}{}^{0.48} \cdot \rho_l{}^{0.17} \cdot \rho_t{}^{-0.17} \cdot \rho_{Pn}{}^{-0.50} - 1 \quad (21)$$

$$\alpha_{uk} = e^{-4.99} \cdot A_g^{1.61} \cdot L^{-1.96} \cdot f'_c{}^{0.24} \cdot f_{yl}{}^{-0.29} \cdot \rho_l{}^{0.08} \cdot \rho_t{}^{-0.17} \cdot \rho_{Pn}{}^{-0.35} - 1 \quad (22)$$

$$\alpha_{st} = e^{4.58} \cdot A_g^{0.70} \cdot L^{-0.79} \cdot f'_c{}^{-1.12} \cdot f_{yl}{}^{-0.24} \cdot \rho_l{}^{0.81} \cdot \rho_t{}^{-0.17} \cdot \rho_{Pn}{}^{-0.48} - 1 \quad (23)$$

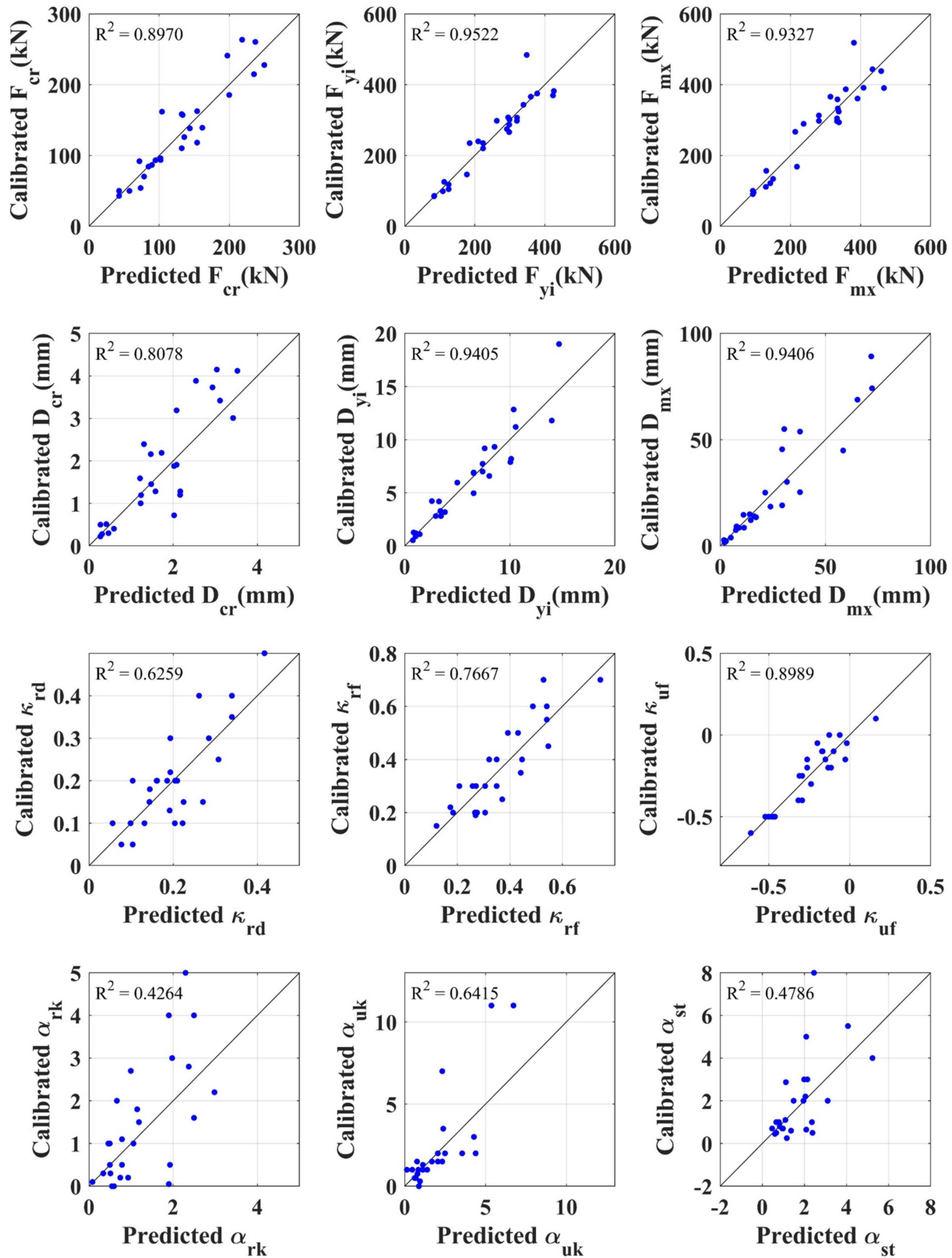


Figure 6. Results of regression analysis.

The R^2 values for the pinching parameters (0.6259 to 0.8989) and the degradation parameters (0.4264 to 0.6415) are relatively lower, suggesting a moderate predictive accuracy. This can be attributed to the inherently nonlinear nature of pinching and degradation behaviors, which are influenced by a multitude of factors, including concrete damage

state, crack width, transverse reinforcement stress, and the opening and closing of concrete cracks [31].

Figure 7 represents a statistical comparison of the errors between predicted and calibrated parameters. The near-zero median error suggests minimal bias in the predictions. Consistent with the high R^2 values observed in the regression analysis, the backbone curve parameters demonstrate low standard deviations, indicative of high prediction precision. However, for the unloading force pinching parameter (κ_{uf}) and degradation parameters (α_{uk} , α_{rk} , α_{st}), the standard deviation values are either close to or exceed 1. This implies greater variability in the prediction errors for these parameters compared to the backbone curve parameters.

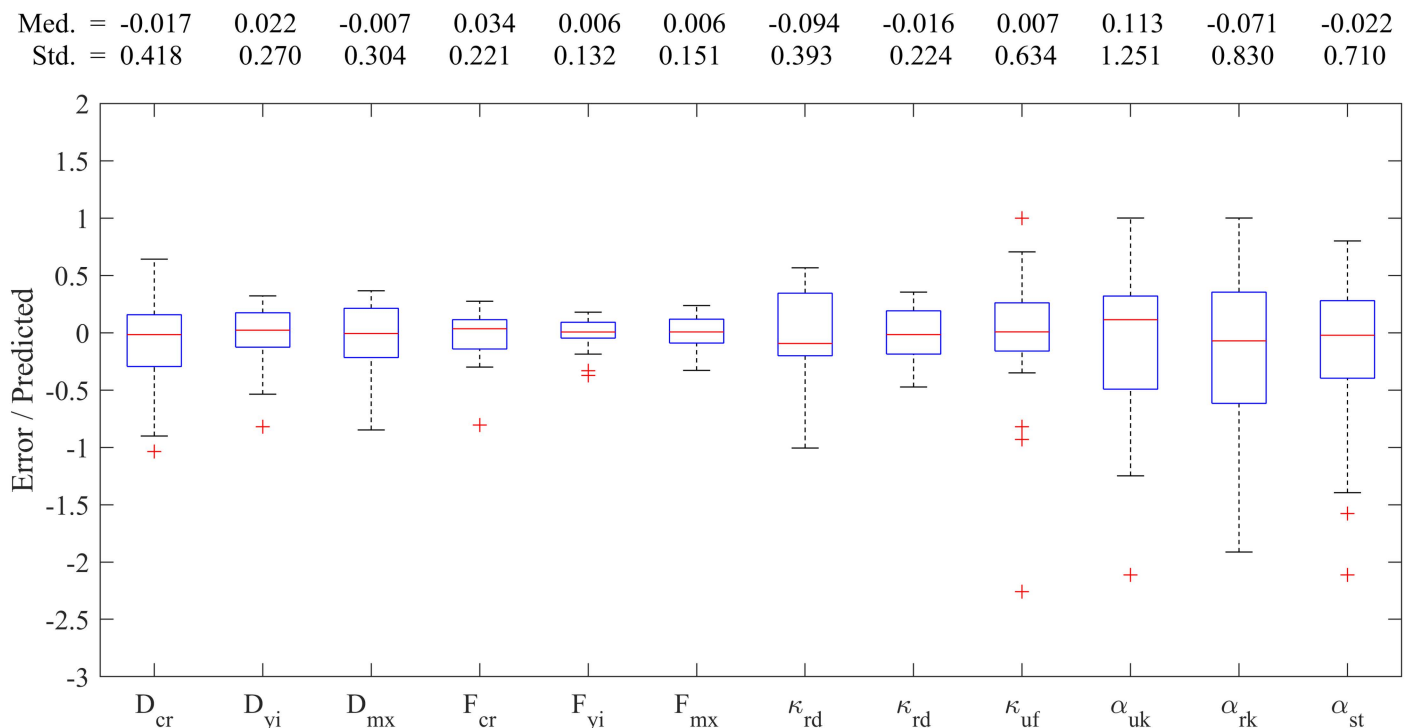


Figure 7. Statistical comparison of the errors between predicted and calibrated parameters. Note: The red cross represents an outlier.

3.4. Verification of Proposed Equations

Figures 8 and 9 illustrate the application of the predicted parameters to the analysis of the collected column specimens. Figure 8 exhibits a close agreement between the predicted and experimental results, underscoring the efficacy of the predictive parameters in capturing the member response. The results using the proposed prediction equations simulate flexural (Figure 8a,d), flexural–shear (Figure 8f), and shear (Figure 8b,c,e) behaviors well. However, as shown in the statistical comparison in Figure 7, differences due to the errors in the degradation parameters were observed, but these errors are not considered significant enough to affect the performance evaluation results. Figure 9, on the other hand, presents cases where the predicted behavior deviates from the experimental results. Figure 9a represents a case where pinching behavior is not adequately captured, potentially due to limitations in predicting pinching parameters. In Figure 9b, the backbone curve parameters are computed with a notably low value, leading to an underestimation of maximum strength. Figure 9c–f show cases where the predicted performance degradation is more rapid than that observed in the experiments, resulting in a lower capacity. These discrepancies can be partly attributed to the higher variability associated with predicting pinching and degradation parameters, as evident in Figures 6 and 7.

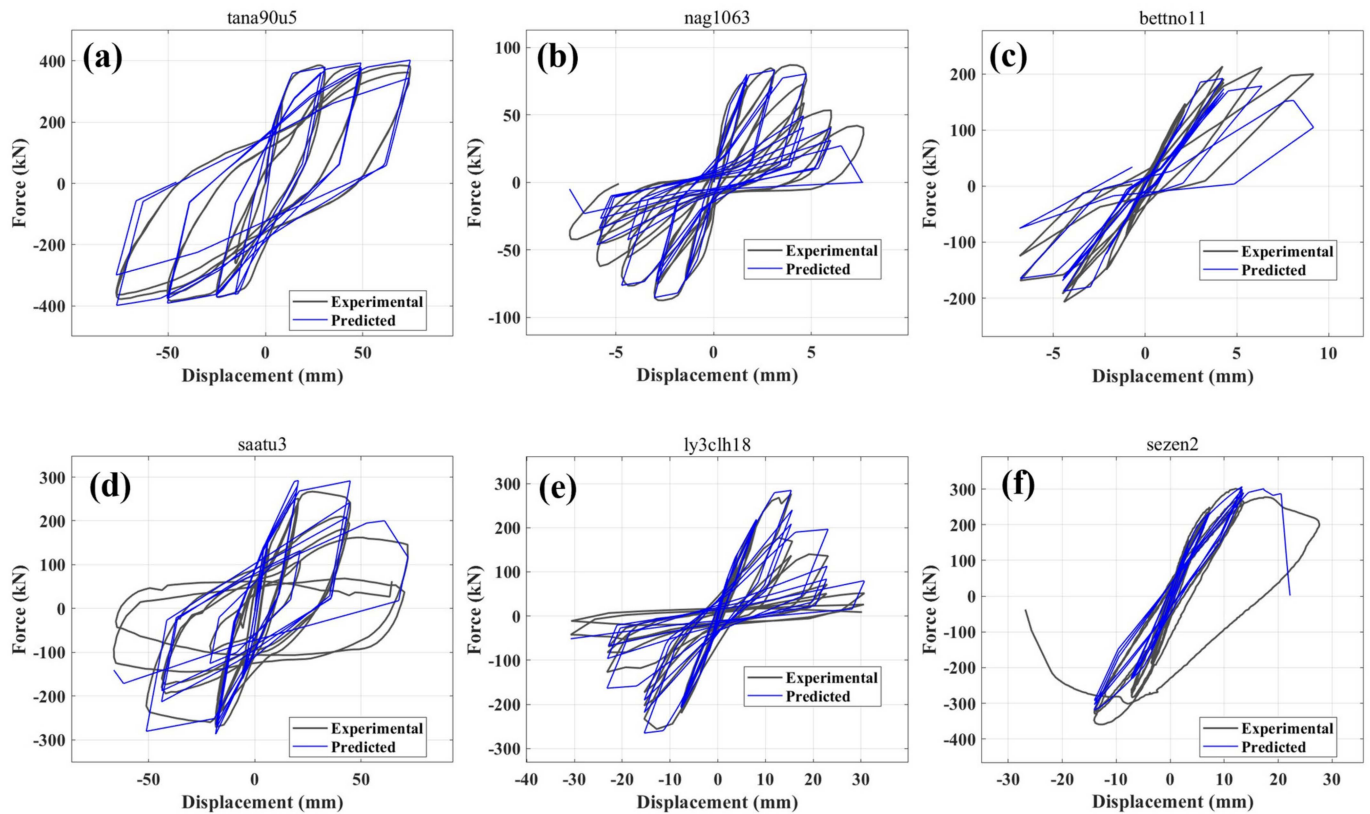


Figure 8. Comparison experimental results (gray solid line) with analytical predictions (blue solid line) using the proposed equations for well-matched cases: (a) tana90u5; (b) nag1063; (c) bettno11; (d) saatu3; (e) ly3clh18; and (f) sezen2.

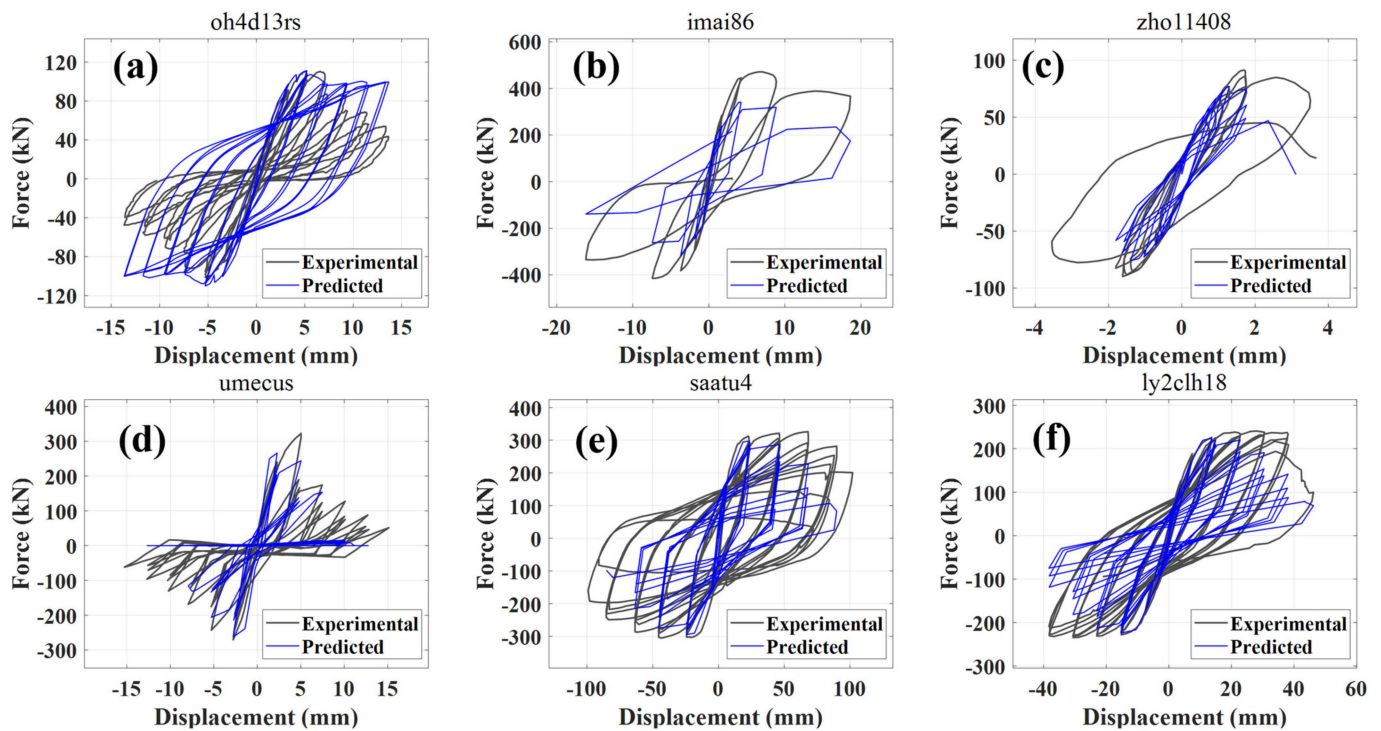


Figure 9. Comparison experimental results (gray solid line) with analytical predictions (blue solid line) using the proposed equations for poorly matched cases: (a) oh4d13rs; (b) imai86; (c) zho11408; (d) umecus; (e) saatu4; and (f) ly2clh18.

Figure 10 and Table 1 present the statistical comparison between the experimental data, the calibrated model, and the predicted model. Many experimental studies on the seismic performance of structural members terminate testing when member strength decreases by 60% to 80% compared to its peak strength. However, for the calculation of performance metrics such as ductility ratio, the ultimate deformation is typically defined as the deformation when the reduced strength reaches 80% to 85% of the maximum strength on the backbone curve [32–36]. Additionally, in cases where significant strength reduction (up to 60%) is observed in the specimen, it is often assumed that the member has failed, and its subsequent behavior has negligible impact on the structural performance. Therefore, in this study, statistical comparisons between experimental and predicted results were primarily focused on data points up to 60 to 80% of the maximum strength of the specimen.

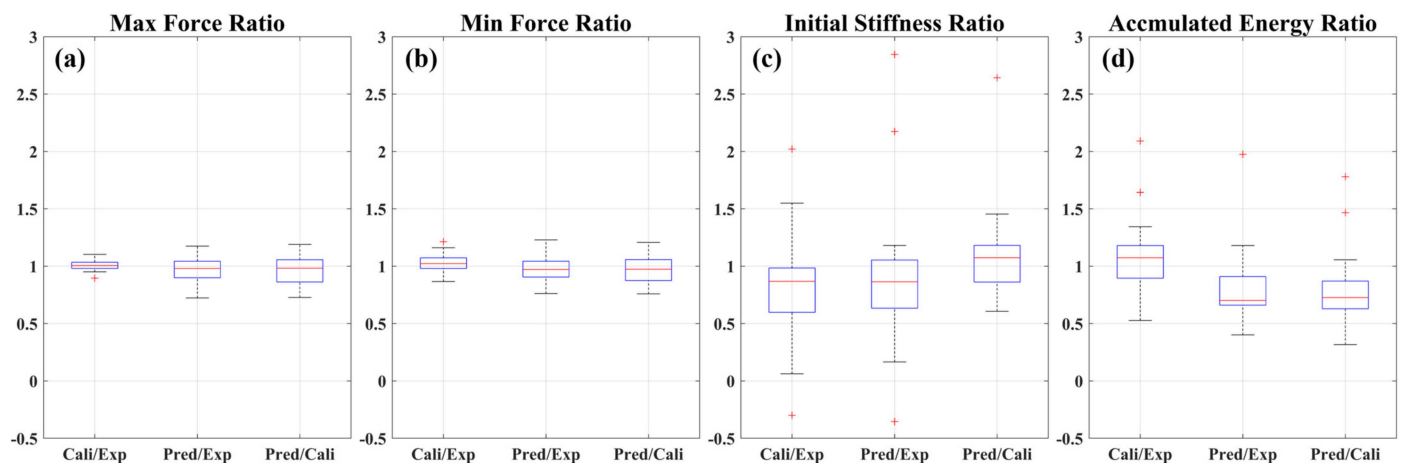


Figure 10. Statistical comparison of experiment results, calibrated models, and predicted models: (a) maximum force ratio; (b) minimum force ratio; (c) initial stiffness ratio; and (d) accumulated energy ratio.

Table 1. Median and standard deviation of models.

		Maximum Force	Minimum Force	Initial Stiffness	Accumulated Energy
Calibrated/Experiment	Med.	1.006	1.023	0.868	1.074
	Std.	0.046	0.075	0.479	0.326
Predicted/Experiment	Med.	0.981	0.971	0.864	0.702
	Std.	0.110	0.126	0.604	0.314
Predicted/Calibrated	Med.	0.983	0.974	1.074	0.727
	Std.	0.117	0.114	0.379	0.317

The calibrated model exhibits good agreement with the experimental data for maximum and minimum forces as shown in Table 1 and Figure 10. The median values for maximum and minimum forces are 1.006 and 1.023, respectively, with relatively low standard deviations of 0.046 and 0.075. However, the median value of 0.868 for initial stiffness suggests a tendency for the calibrated model to underestimate the initial stiffness. The median for accumulated energy is 1.074, closely approaching unity.

The predicted model, based on Equations (12)–(23), shows similar trends to the calibrated model for maximum and minimum forces, with median values of 0.981 and 0.971, respectively, when compared to the experiment data. However, the predicted model exhibits a more pronounced underestimation of accumulated energy, with a median value of

0.702, compared to the experimental result. This discrepancy can be attributed to the higher variability observed in the pinching and gradation parameters as highlighted in Figure 7. Although the median values for these parameters in the statistical comparison of the errors between predicted and calibrated parameters are close to zero, the relatively large standard deviation of some parameters (κ_{uf} , α_{uk} , α_{rk} , α_{st}) contributes to this observation.

3.5. Comparison with Existing Models

To evaluate the predictive accuracy of the proposed model and existing models, the ratio of analytical results to experimental data was determined for three key parameters: maximum force (R_F), initial stiffness (R_{K0}), and cumulative energy dissipation capacity ($R_{\Sigma E}$). The performance of the proposed model was compared against those of Lee and Han (2018) [16] and Haselton et al. (2016) [21] using data from 15 column specimens as shown in Table 2 and Figure 11.

Table 2. Median and standard deviation of comparative analysis between proposed and existing models.

Models	Maximum Force (R_F)			Initial Stiffness (R_{K0})			Cumulated Energy Dissipation Capacity ($R_{\Sigma E}$)		
	Proposed	Lee and Han (2018) [16]	Haselton et al. (2016) [21]	Proposed	Lee and Han (2018) [16]	Haselton et al. (2016) [21]	Proposed	Lee and Han (2018) [16]	Haselton et al. (2016) [21]
Median	0.96	0.97	0.95	0.94	0.76	0.83	0.82	0.74	0.94
Standard Deviation	0.11	0.11	0.24	0.25	0.22	0.35	0.13	0.19	0.26

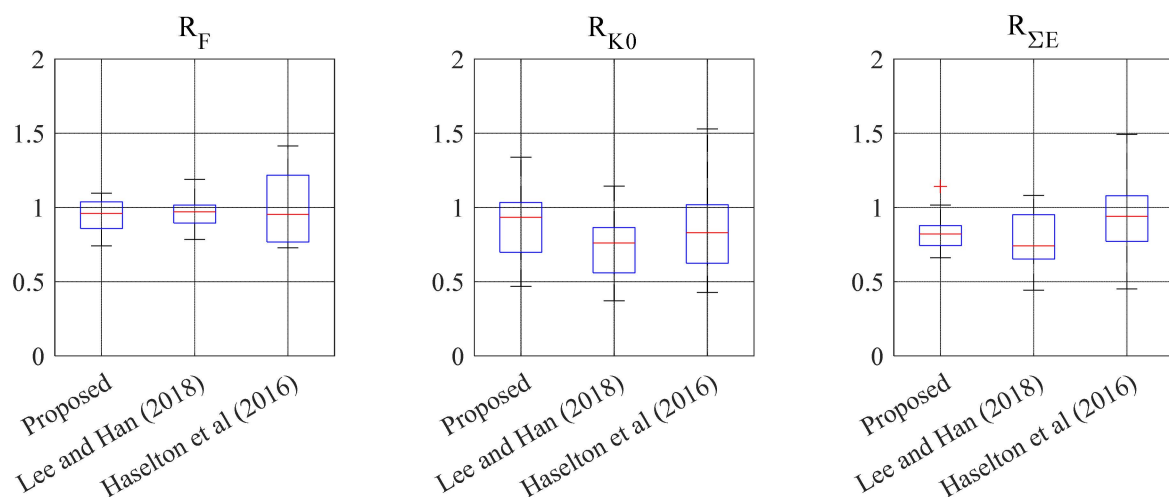


Figure 11. Statistical comparison with existing models [16,21]. Note: red cross means outlier.

For maximum force prediction, all three models exhibit strong agreement with experimental data, with a median value of 0.95. In terms of standard deviation, the proposed approaches including those from Lee achieve the lowest value at 0.11, indicating a high degree of prediction precision. In terms of initial stiffness prediction, the proposed model demonstrates the most accurate results with a median value of 0.94 and a relatively low standard deviation of 0.25. Regarding cumulative energy dissipation, the Haselton approach exhibited the closest agreement to experimental data in terms of the median value. However, its higher standard deviation compared to the other approaches indicates a lower level of prediction consistency. Therefore, based on the experimental data considered in this study, it could be concluded that the proposed model offers better predictive performance for maximum force, initial stiffness, and cumulative energy dissipation compared to existing models.

4. Seismic Fragility Assessment

4.1. Selected Structure

To investigate the impact of shear deformation in non-ductile RC columns on the seismic fragility of RC structures, a non-conforming building representative of the standard drawings for 1980s school buildings provided by the Ministry of Education in South Korea was selected. The proposed prediction equations for shear spring model parameters, detailed in the previous section, were employed to account for the shear deformation of RC columns. Figure 12 illustrates the plan view and elevation of the selected RC structure. For simplicity, a frame in the short direction was selected, as the aim of this study is to compare the seismic behavior of RC frames with and without considering the shear deformation of RC columns.

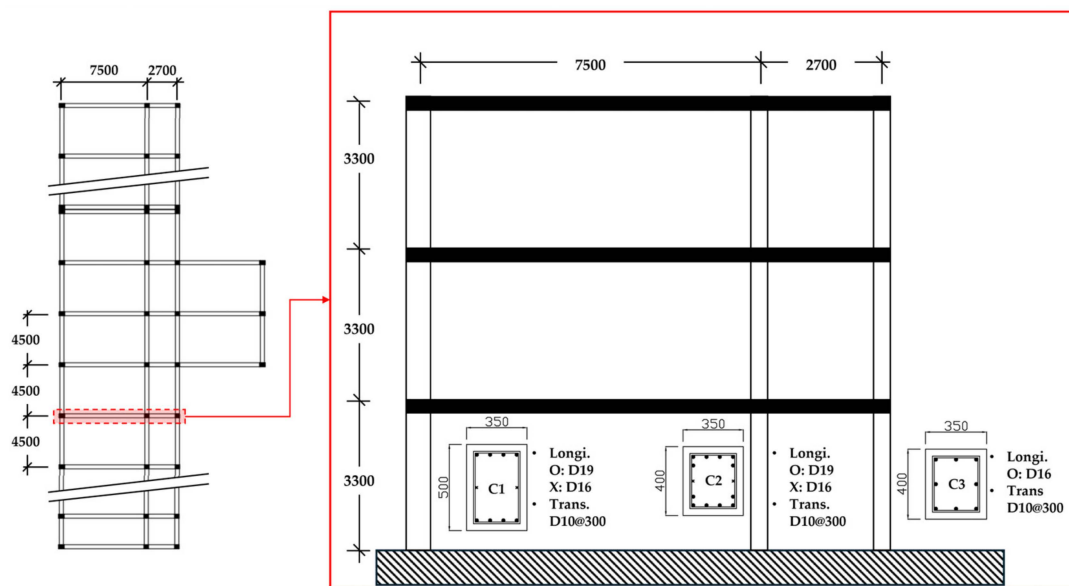


Figure 12. Plan view and elevation of the selected RC structure (unit: mm).

The section details of RC columns are shown in Figure 13. Columns C1, C2, and C3 have rectangular cross-sections with dimensions of 350 mm × 500 mm, 350 mm × 400 mm, and 350 mm × 400 mm, respectively. The quantity and diameter of longitudinal reinforcement vary: C1 contains 8-D19 and 2-D16 rebars, C2 has 12-D19 and 2-D16 rebars, and C3 is reinforced with 8-D16 rebars. These configurations result in longitudinal reinforcement ratios of 1.54% for C1, 2.74% for C2, and 1.13% for C3. Additionally, the volumetric ratios of transverse reinforcement for these three columns are 0.30%, 0.34%, and 0.34%, respectively, indicating relatively low confinement. In accordance with the 1980s specifications for school buildings in South Korea, the compressive strength of concrete and the yield strength of the rebar are assumed to be 21 MPa and 300 MPa, respectively.

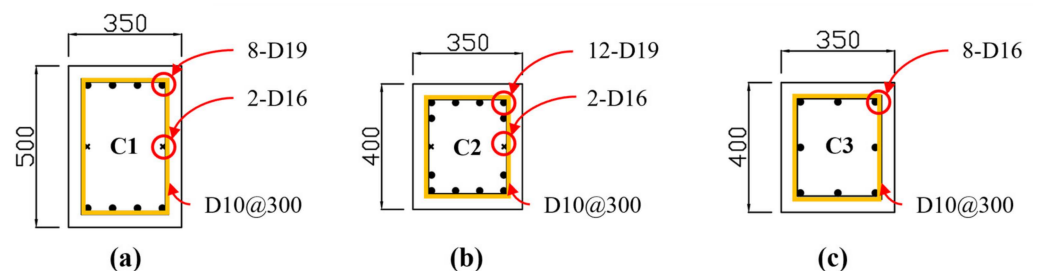


Figure 13. Section details of RC columns (unit: mm): (a) C1; (b) C2; and (c) C3.

4.2. Analytical Models

Analytical models of the RC structure were implemented and analyzed using OpenSees. As illustrated in Figure 14, two analytical models were considered: one incorporating shear springs and one without. The RC frame members were modeled using displacement-based beam–column elements, employing the material constitutive laws of Concrete02 [37] for concrete and Steel02 [38] for rebars, respectively. For the confined concrete, the confinement effect on the stress and strain was considered by utilizing the formulae developed by Cusson and Paultre (1994) [30]. In the first analytical model, to account for the shear deformation in RC columns, shear springs were added to both ends of each column. These springs were modeled using zero-length elements with the Pinching4 material model. To isolate the effect of shear springs in columns, the beam elements were assumed to be rigid in both models.

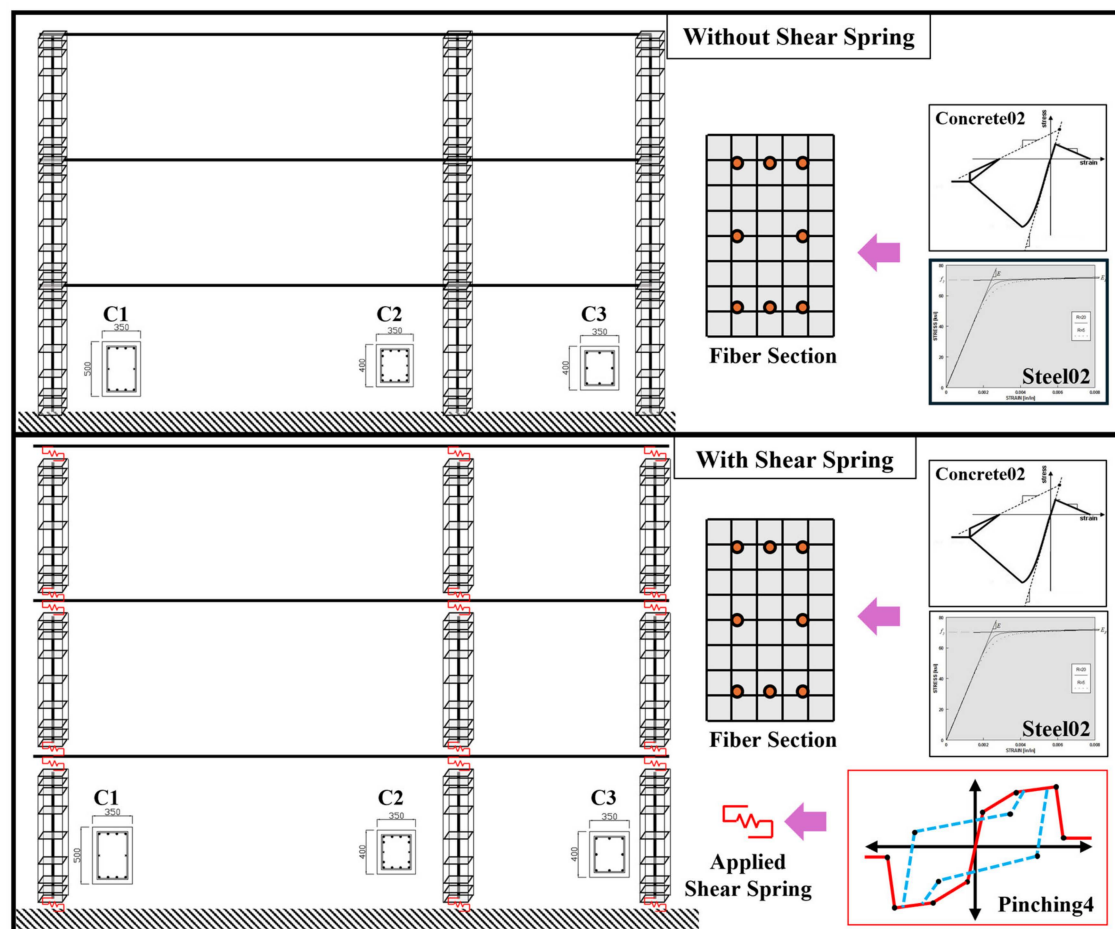
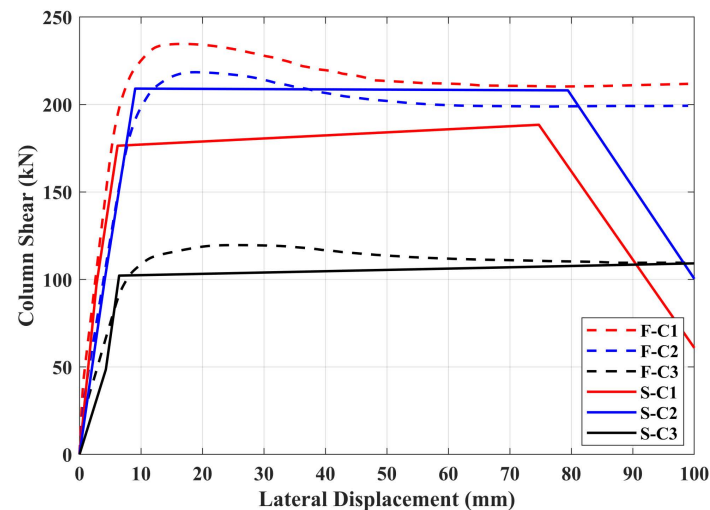


Figure 14. Analytical models with and without shear springs.

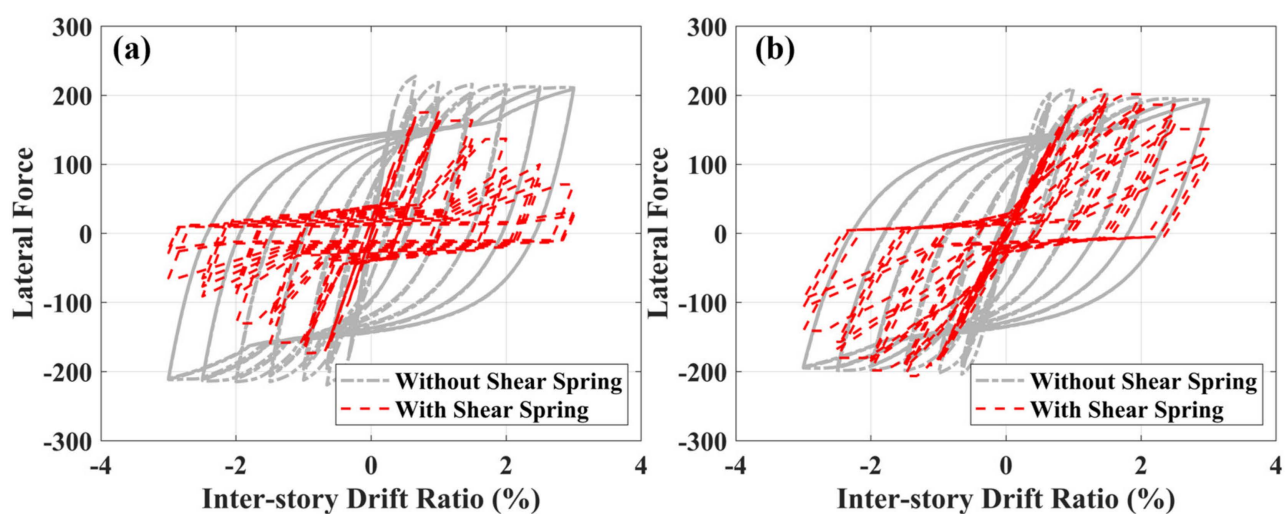
The Pinching4 model parameters for the shear springs were estimated using the equations proposed in the previous section. Table 3 summarizes the predicted parameters for the three columns, and Figure 15 compares the resulting backbone curves to the pushover analysis of the flexure model. Although the backbone curves generally resemble the pushover results, column C1 exhibits a significantly lower predicted shear strength, indicating a dominance of shear behavior. In contrast, the shear strengths of columns C2 and C3 closely match the maximum lateral force of the flexural model, suggesting that these columns exhibit flexural or flexural–shear behavior.

Table 3. Predicted spring parameters for RC columns.

Specimen		D_{cr} (mm)	D_{yi} (mm)	D_{mx} (mm)	F_{cr} (kN)	F_{yi} (kN)	F_{mx} (kN)	κ_{rd}	κ_{rf}	κ_{uf}	α_{uk}	α_{rk}	α_{st}
No	Name												
1	C1	2.77	6.20	74.72	97.14	176.45	188.42	0.45	0.27	0.20	2.21	5.97	6.92
2	C2	4.08	9.06	79.45	99.26	209.10	208.10	0.38	0.15	0.04	0.96	4.13	7.23
3	C3	4.29	6.45	138.94	48.57	102.22	112.12	0.65	0.31	0.46	1.92	7.79	6.64

**Figure 15.** Comparison between pushover analysis with the flexure model and shear spring backbone curve.

To investigate the effects of shear deformation, cyclic analyses of RC columns with and without the shear spring were conducted, as shown in Figure 15. The loading pattern consisted of interstory drift ratios (IDRs) of 0.33%, 0.66%, 1%, 1.5%, 2%, 2.5%, and 3%, with three cycles applied at each level. As expected from Figure 15, column C1 exhibits shear-dominant behavior, as shown in Figure 16a. The column's strength rapidly deteriorates after an IDR of 1%, accompanied by a clear degradation in reloading and unloading stiffness due to the cyclic loading. As shown in Figure 16b, column C2 demonstrates that strength degradation begins at an IDR of 1.5% and accelerates after an IDR of 2%.

**Figure 16.** Comparison of cyclic behavior of RC columns with and without shear spring: (a) C1; (b) C2.

4.3. Selected Ground Motions and Limit States

One hundred earthquake ground motion records were selected from the PEER Center strong motion database [39] for the seismic fragility assessment. These ground motions

aimed to represent a range of earthquake intensities. Specifically, they included moment magnitudes (M_w) between 5 and 8 and close distances to the fault rupture ranging from 3 to 40 km. Figure 17 illustrates the acceleration response spectrum of selected records. The spectral accelerations (S_a) are distributed relatively evenly across a range of 0.01 g to 2.10 g at the fundamental period (0.26 s) of the selected structure, as determined through eigenvalue analysis.

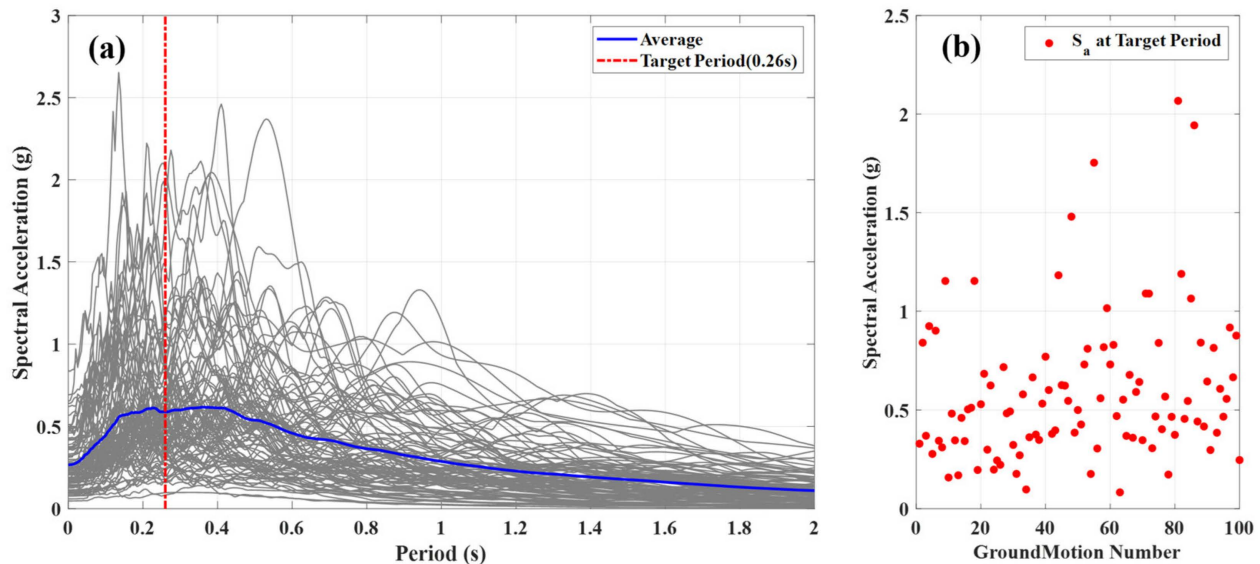


Figure 17. Selected ground motions: (a) acceleration response spectra; (b) distribution of S_a at the fundamental period of the selected structure.

For the selected structure, this study adopts three limit states associated with the structural performance levels, as defined by Kwon and Elnashai (2006) [40]: serviceability, damage control, and collapse prevention. Limit states are defined separately for models with and without shear springs. For the conventional model without shear springs, the serviceability limit state is defined when longitudinal reinforcement reaches yielding stress. The damage control state is defined as the point at which the column experiences its maximum moment. The collapse prevention state is reached when the concrete strain equals the ultimate confined strain, as suggested by Eurocode 8 [41]. The IDR for the three limit states, selected according to the above criteria, are 0.37%, 0.83%, and 1.05%, respectively.

However, the aforementioned limit state definition is only applicable when the stress–strain relationship of each material fiber is known in the numerical model. When a shear spring is applied, this relationship is unknown, so the limit states for the model with shear springs are defined in the same way as for the flexural model, with the addition of a shear failure state. Since the degradation parameter of the spring element cannot be considered when defining the limit state using the pushover analysis results, as in the existing model, the results of the cyclic analysis with the loading protocol shown in Figure 15 were used. Extensive studies [13,42–45] have proposed the point at which the idealized backbone curve reaches the shear strength as the point of shear failure. Therefore, in this study, the IDR at which the shear strength is first reached is also defined as the shear failure point. As shown in the results in Figure 15, the IDR when the shear spring first reaches the shear strength is 0.7%, so the IDR for the shear limit state is defined as 0.70%.

4.4. Seismic Fragility Curves

Figure 18 illustrates the maximum IDR obtained from nonlinear dynamic analyses of two analytical models subjected to a suite of selected ground motion records. For the model

without shear springs, the first story exhibits the highest mean IDR of 0.332%, closely followed by the second story with the IDR of 0.331%. Median IDRs show a marginal increase for the second story (0.294%) compared to the first story (0.274%). In the model with shear springs, the second story demonstrates the highest mean IDR, reaching 0.554%, while the first story follows closely at 0.553%. Median IDRs also indicate the second story as the most critical, with a value of 0.534% compared to 0.508% for the first story. Notably, the inclusion of shear springs results in IDRs more than twice as large as those observed in the model without shear springs under identical seismic inputs. This highlights the substantial influence of shear spring implementation on the structural performance evaluation.

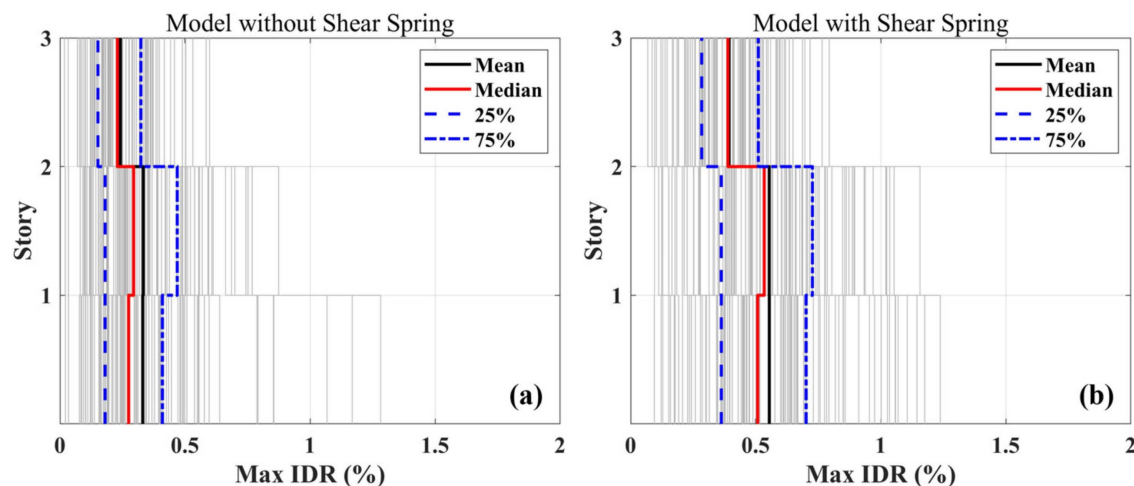


Figure 18. Maximum IDR for each story: (a) non-shear model; (b) shear model.

This study conducted seismic fragility assessments using the spectral acceleration of selected ground motions as the intensity measure and IDR derived from nonlinear time history analysis as the engineering demand parameter. Seismic fragility curves were developed through linear regression analysis, utilizing a natural logarithmic scale for the engineering demand parameter and intensity measure. Figure 19 provides a visual representation of the seismic fragility curves, highlighting the differences between models with and without shear springs. Figure 19a depicts the probability of exceeding various damage levels for both models. Figure 19b specifically highlights the differences in these probabilities between the two models (shear spring model minus non-shear spring model). Figure 19b reveals that, across all damage states, the model without shear springs consistently exhibits lower probabilities of exceedance compared to the model with shear springs. The most significant difference in the probability of exceeding a given damage level is observed when shear failure is assumed to correspond to the collapse prevention limit state at the spectral acceleration of 1.30 g, resulting in a 58.24% variation. In the serviceability limit state, a 48.80% difference occurs at 0.44 g. For the damage control limit state, a 27.83% difference is observed at 1.06 g, while the collapse prevention limit state alone shows a 21.68% difference at 1.32 g. These results indicate that the limit state most affected by the presence or absence of shear spring is the collapse prevention state, with a significant impact. Additionally, the substantial differences observed not only in the collapse prevention limit state but also in the serviceability limit state demonstrate the considerable influence of shear springs on the overall structural stability. These findings indicate that performance evaluations relying only on fiber-section-based elements significantly overestimate the seismic performance of the structure.

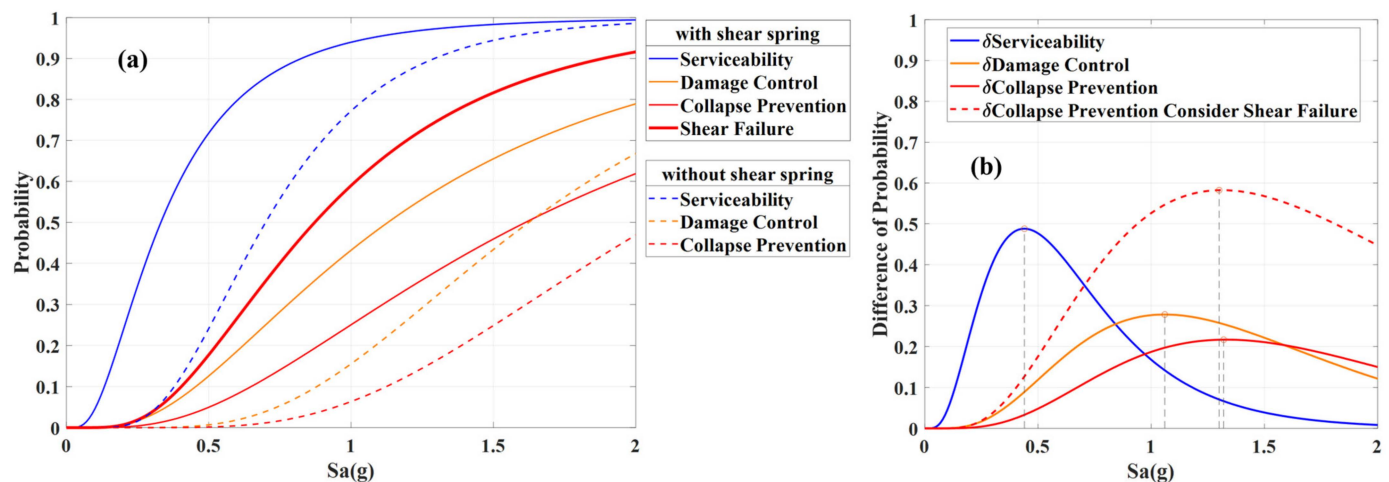


Figure 19. Seismic fragility curves for shear and flexural models: (a) seismic fragility curves; (b) difference of probability in individual limit states.

5. Conclusions

This paper presents predictive equations for shear spring parameters that can be incorporated into fiber-section elements to more accurately capture the behavior of RC columns with non-conforming section details. In addition, seismic fragility assessments were conducted on an example structure utilizing the proposed shear spring model. The most important findings are summarized below.

Predictive equations for shear spring parameters, encompassing shear behavior, cyclic degradation, and pinching effects, were developed based on the physical properties of RC columns. While the predictions for backbone curve parameters exhibited good accuracy, those for pinching and degradation parameters showed relatively lower accuracy.

Comparative analysis with existing models revealed superior performance of the proposed model in predicting key hysteretic parameters, including peak force, initial stiffness, and cumulative energy dissipation. Notably, the highest accuracy and consistency were observed for peak force and initial stiffness predictions, demonstrated by the lowest standard deviations.

Seismic fragility assessments of non-conforming RC frames revealed a significant disparity in damage probabilities between models with and without shear springs. Models without shear springs tended to overestimate structural performance, emphasizing the crucial role of shear springs in accurately assessing the seismic performance of non-conforming RC frames.

Therefore, the proposed parameter prediction equations enable the simulation of the shear behavior of non-conforming RC columns with reasonable accuracy, forming a basis for more realistic seismic performance evaluations of RC frames. Furthermore, these equations could have practical applications in guiding the development of more reliable design and retrofitting strategies for non-conforming RC structures.

However, the proposed model is based on a limited dataset of experimental results, which may constrain its predictive accuracy. Future research should prioritize expanding the experimental database, particularly for RC columns with diverse geometric properties. Additionally, refining data collection methodologies and incorporating advanced simulation techniques could further enhance the model's robustness and applicability. These efforts will be crucial for elevating the proposed model's utility and ensuring its practical contribution to the fields of structural performance evaluation and seismic design.

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Abbreviations

The following abbreviations are used in this manuscript:

RC	reinforced concrete
PGA	peak ground acceleration
SA	spectral acceleration
PEER	Pacific Earthquake Engineering Research
R	rectangular
RDs	rectangular and diagonal ties
RJs	rectangular ties with J-hooks
C	cantilever
DC	double curvature
F	flexural failure
FS	flexural–shear failure
S	shear failure
IDR	interstory drift ratio

Appendix A

Table A1. Properties of selected columns from the PEER database.

Specimen		B (mm)	H (mm)	L (mm)	f'_c (MPa)	f_{yl} (MPa)	f_{yt} (MPa)	ρ_l	ρ_t	P_n (kN)	Failure Mode
No.	Name										
1	tana90u5	550.0	550.0	1650.0	32.0	511.0	325	0.012	0.032	968	F
2	tana90u7 *	550.0	550.0	1650.0	32.1	511.0	325	0.012	0.039	2913	F
3	nag1063	200.0	200.0	300.0	21.6	371.0	344	0.013	0.018	147	S
4	oh2d16rs	200.0	200.0	400.0	32.0	369.0	316	0.020	0.013	183	FS
5	oh4d13rs	200.0	200.0	400.0	29.9	370.0	316	0.027	0.012	183	FS
6	imai86	400.0	500.0	825.0	27.1	318.0	336	0.027	0.007	392	S
7	zho10408	160.0	160.0	160.0	19.8	341.0	559	0.022	0.016	406	S
8	zho11408	160.0	160.0	160.0	19.8	341.0	559	0.022	0.016	406	S
9	zho12408 *	160.0	160.0	160.0	19.8	341.0	559	0.022	0.039	406	S
10	zho20408 *	160.0	160.0	320.0	21.1	341.0	559	0.022	0.016	432	FS
11	zho21408 *	160.0	160.0	320.0	21.1	341.0	559	0.022	0.016	432	F
12	zho22309 *	160.0	160.0	320.0	21.1	341.0	559	0.022	0.039	486	FS
13	zho30207 *	160.0	160.0	480.0	28.8	341.0	559	0.022	0.016	517	FS
14	zho31207 *	160.0	160.0	480.0	28.8	341.0	559	0.022	0.016	517	FS
15	zho32207 *	160.0	160.0	480.0	28.8	341.0	559	0.022	0.039	517	FS
16	kanstc1 *	250.0	250.0	750.0	27.9	374.0	506	0.016	0.008	184	F
17	kanstc2 *	250.0	250.0	750.0	27.9	374.0	506	0.016	0.008	184	F

Table A1. Cont.

Specimen		B (mm)	H (mm)	L (mm)	f'_c (MPa)	f_{yl} (MPa)	f_{yt} (MPa)	ρ_l	ρ_t	P_n (kN)	Failure Mode
No.	Name										
18	kanstc3 *	250.0	250.0	750.0	27.9	374.0	506	0.016	0.008	184	F
19	ara89oa2	180.0	180.0	225.0	31.8	340.0	249	0.031	0.006	190	S
20	ara89oa5	180.0	180.0	225.0	33.0	340.0	249	0.031	0.006	476	S
21	umecus	230.0	410.0	455.0	34.9	441.0	414	0.030	0.011	534	S
22	umecuw	410.0	230.0	455.0	34.9	441.0	414	0.030	0.010	534	S
23	bettno11	305.0	305.0	457.0	29.9	462.0	414	0.024	0.006	288	S
24	saatu3	350.0	350.0	1000.0	34.8	430.0	470	0.032	0.016	600	F
25	saatu4	350.0	350.0	1000.0	32.0	438.0	470	0.032	0.023	600	F
26	ly3clh18	457.2	457.2	1473.2	26.9	331.0	399.9	0.030	0.002	503	S
27	ly2clh18	457.2	457.2	1473.2	33.1	331.0	399.9	0.019	0.002	503	FS
28	ly2cmh18	457.2	457.2	1473.2	25.5	331.0	399.9	0.019	0.002	1512	FS
29	ly3cmh18	457.2	457.2	1473.2	27.6	331.0	399.9	0.030	0.002	1512	S
30	ly3cmd12	457.2	457.2	1473.2	27.6	331.0	399.9	0.030	0.006	1512	S
31	ly3slh18	457.2	457.2	1473.2	26.9	331.0	399.9	0.030	0.002	503	S
32	ly2slh18	457.2	457.2	1473.2	33.1	331.0	399.9	0.019	0.002	503	FS
33	ly3smd12	457.2	457.2	1473.2	25.5	331.0	399.9	0.030	0.006	1512	FS
34	moc11 *	400.0	400.0	1400.0	24.9	497.0	459.5	0.021	0.026	450	F
35	moc12 *	400.0	400.0	1400.0	26.7	497.0	459.5	0.021	0.026	675	F
36	moc13 *	400.0	400.0	1400.0	26.1	497.0	459.5	0.021	0.026	900	F
37	sezen1	457.2	457.2	1473.2	21.1	434.4	476	0.025	0.005	667	FS
38	sezen2	457.2	457.2	1473.2	21.1	434.4	476	0.025	0.005	2669	FS
39	sezen4	457.2	457.2	1473.2	21.8	434.4	476	0.025	0.005	667	FS
40	Takemura_Kawashima3	400.0	400.0	1245.0	34.3	363.0	368	0.016	0.005	157	F

b : width of column section; h : height of column section; L : column length; f'_c : concrete compressive strength; f_{yl} , f_{yt} : yield stress of longitudinal and transverse rebars; ρ_l , ρ_t : longitudinal and transverse reinforcement ratios; P_n : applied axial load. Note: * denotes data excluded from the regression analysis.

Table A2. Calibrated shear spring parameters for selected specimens.

Specimen		D_{cr} (mm)	D_{yi} (mm)	D_{mx} (mm)	F_{cr} (kN)	F_{yi} (kN)	F_{mx} (kN)	κ_{rd}	κ_{rf}	κ_{uf}	α_{uk}	α_{rk}	α_{st}
No *	Name												
1	tana90u5	3.88	9.19	68.82	263.49	366.10	443.46	0.50	0.70	−0.15	1.50	0.50	0.70
3	nag1063	0.40	1.10	3.89	49.97	86.18	95.61	0.20	0.20	−0.10	1.00	1.00	0.80
4	oh2d16rs	1.28	4.18	14.63	54.09	99.20	111.52	0.13	0.20	−0.05	0.75	0.30	0.45
5	oh4d13rs	1.45	3.29	8.58	70.32	104.97	121.65	0.18	0.30	−0.30	1.50	1.00	1.10
6	imai86	0.30	2.81	13.97	240.98	483.95	518.44	0.15	0.35	−0.20	11.00	2.20	4.00
7	zho10408	0.22	0.90	1.59	43.02	85.34	90.98	0.05	0.30	−0.50	1.30	0.50	1.00
8	zho11408	0.50	1.20	2.59	50.10	84.83	100.44	0.20	0.40	−0.50	1.00	1.10	0.50
19	ara89oa2	0.51	1.27	2.23	91.86	125.51	156.70	0.05	0.15	−0.50	2.00	0.20	1.00
20	ara89oa5	0.28	0.53	2.76	84.37	118.08	133.96	0.10	0.22	−0.60	3.50	0.10	2.87
21	umecus	1.20	3.18	7.47	227.77	301.33	387.18	0.20	0.20	−0.05	3.00	1.80	3.00
22	umecuw	1.00	2.81	8.55	93.29	240.05	289.49	0.15	0.19	0.00	2.00	1.50	2.20
23	bettno11	2.19	4.22	9.18	161.81	234.97	267.01	0.10	0.20	−0.15	11.00	2.80	2.00
24	saatu3	4.15	19.00	44.90	138.36	274.56	293.70	0.10	0.60	−0.10	0.50	0.00	0.70
25	saatu4	1.28	11.80	89.20	126.06	307.50	358.49	0.10	0.45	−0.10	1.00	0.30	0.70
26	ly3clh18	3.19	7.00	19.10	158.40	287.10	304.70	0.30	0.30	−0.15	1.50	1.60	5.50
27	ly2clh18	0.72	6.91	25.30	93.67	235.00	313.00	0.35	0.55	−0.25	1.50	0.05	0.65
28	ly2cmh18	1.59	5.96	14.90	157.20	298.00	366.00	0.10	0.40	−0.50	0.30	0.20	0.25
29	ly3cmh18	2.16	6.84	13.50	139.20	343.20	360.80	0.20	0.25	−0.50	1.00	2.70	2.00
30	ly3cmd12	3.42	11.20	18.50	214.80	369.60	390.50	0.25	0.50	−0.25	0.50	0.00	0.60
31	ly3slh18	1.91	7.73	45.50	110.28	266.20	297.00	0.22	0.20	−0.20	2.00	4.00	5.50
32	ly2slh18	1.88	4.96	53.80	96.00	220.00	297.70	0.40	0.60	−0.40	7.00	4.00	5.00
33	ly3smd12	3.73	7.89	25.09	260.38	382.02	438.35	0.30	0.50	−0.40	0.50	2.00	2.00
37	sezen1	3.01	8.20	55.02	118.18	306.84	332.77	0.40	0.30	−0.20	1.00	3.00	8.00

Table A2. Cont.

Specimen		D_{cr} (mm)	D_{yi} (mm)	D_{mx} (mm)	F_{cr} (kN)	F_{yi} (kN)	F_{mx} (kN)	κ_{rd}	κ_{rf}	κ_{uf}	α_{uk}	α_{rk}	α_{st}
No *	Name												
38	sezen2	1.20	9.32	12.14	185.46	375.07	391.29	0.20	0.40	−0.50	1.00	1.00	1.00
39	sezen4	4.12	12.84	30.15	162.58	297.91	324.04	0.15	0.30	0.00	0.00	0.50	0.50
40	Takemura Kawashima3	2.40	6.58	74.18	86.63	146.40	168.42	0.20	0.70	0.10	2.00	5.00	3.00

D_{cr} : displacement of cracking point; D_{yi} : displacement of yield point; D_{mx} : displacement of maximum point; F_{cr} : force of cracking point; F_{yi} : force of yield point; F_{mx} : force of maximum point; κ_{rd} : reloading displacement pinching parameter; κ_{rf} : reloading force pinching parameter; κ_{uf} : unloading force pinching parameter; α_{uk} : unloading stiffness degradation parameter; α_{rk} : reloading stiffness degradation parameter; α_{st} : strength degradation parameter. * The specimen numbers are identical to those listed in Table A1.

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