

Article

Nonlinear Finite Element Investigation of Steel–Concrete Interface Effects on the Cyclic Behavior of Steel-Reinforced Concrete (SRC) Columns

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Abstract

The cyclic response of Steel-Reinforced Concrete (SRC) columns is governed by the interaction between the embedded structural steel section and the surrounding concrete. However, most nonlinear finite element studies assume a perfect bond at the steel–concrete interface, neglecting interface slip that may considerably influence the structural response. This study investigates the effects of steel–concrete interface conditions on the cyclic behavior of SRC columns through three-dimensional nonlinear finite element analyses conducted in ABAQUS. The developed modeling approach was first validated against experimental results available in the literature and demonstrated excellent agreement with the measured global response. A comprehensive parametric investigation was subsequently performed by considering two interface conditions (perfect bond and frictional contact), three axial load ratios, four interface friction coefficients, and different structural steel ratios under displacement-controlled cyclic loading. Structural performance was evaluated in terms of hysteretic response, lateral load capacity, stiffness degradation, and cumulative energy dissipation. The results indicate that the perfect bond assumption consistently overestimates the seismic performance of SRC columns by predicting higher lateral strength, greater stiffness, and significantly larger energy dissipation than the frictional interface model. Depending on the axial load ratio, the cumulative energy dissipation capacity of the frictional model was 44.2–73.1% lower than that of the perfect bond model, while the influence of interface modeling became increasingly pronounced at higher axial load levels. Increasing the interface friction coefficient enhanced the composite action between steel and concrete, although its influence on peak lateral resistance remained relatively limited. The findings demonstrate that realistic steel–concrete interface modeling is essential for reliable nonlinear finite element simulations and provides practical guidance for the seismic assessment and design of SRC columns.



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1. Introduction

Steel-encased reinforced concrete (SRC) members are composite structural systems formed by embedding structural steel sections within concrete. SRC members are widely employed in modern structural engineering because of their high load-bearing capacity, ductility, stiffness, and energy dissipation capacity. The composite action between the embedded structural steel and surrounding reinforced concrete enables SRC members to achieve higher strength and more stable cyclic response than conventional reinforced

concrete members. These characteristics make SRC systems particularly suitable for high-rise buildings, bridges, and structures located in regions of high seismicity [1]. Furthermore, the interaction between the embedded structural steel and surrounding concrete contributes to improved confinement and deformation capacity.

The practical application of SRC members is particularly prominent in high-rise and super-high-rise structural systems, where high axial-load demands must be accommodated together with stringent requirements for lateral stiffness and seismic resistance. SRC mega-columns have been employed as primary load-bearing components in several high-rise structural systems reported in the literature [2,3]. In such applications, structural steel sections embedded within reinforced concrete enable the complementary mechanical characteristics of steel and concrete to be utilized within a single composite member. Figure 1 schematically illustrates a representative application of SRC mega-columns in a high-rise structural system together with the typical cross-sectional configuration of an SRC column.

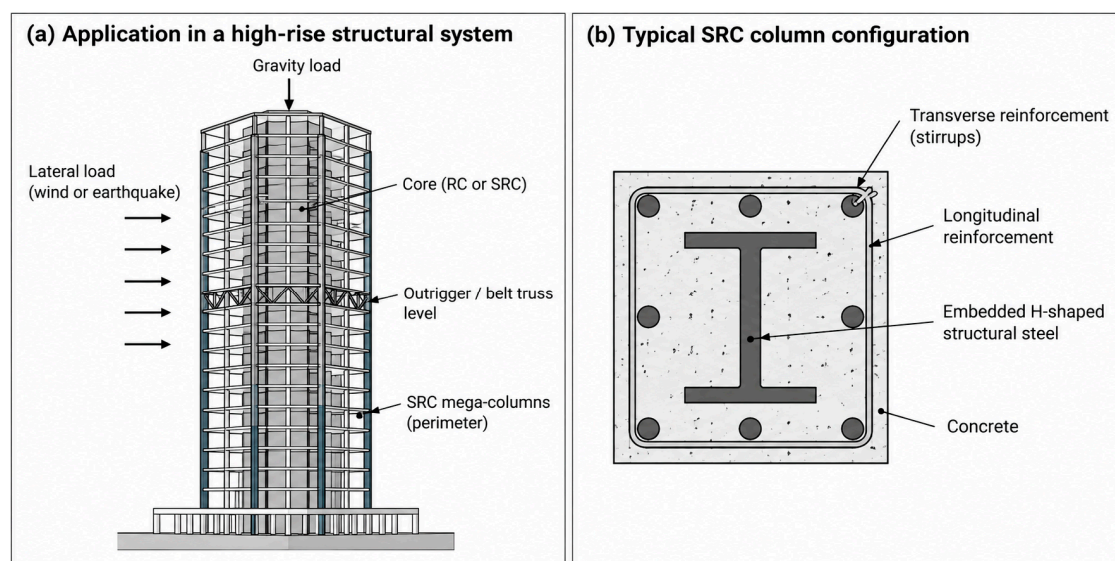


Figure 1. Representative application and configuration of SRC members: (a) schematic application of SRC mega-columns in a high-rise structural system and (b) typical SRC column cross-section with an embedded H-shaped structural steel section, longitudinal reinforcement, and transverse reinforcement. The schematic representation is based on representative SRC applications reported in the literature [1–3].

In recent years, numerous experimental and numerical studies have been conducted to investigate the cyclic behavior of SRC columns. These studies show that the axial load ratio, longitudinal reinforcement ratio, transverse reinforcement detailing, and steel cross-section geometry are the key parameters that determine the seismic performance of SRC columns. Liu et al. [4] investigated the cyclic behavior of SRC columns with T- and L-shaped cross-sections and reported that the axial load ratio and geometric parameters have significant effects on hysteresis behavior, ductility, and failure mechanisms. Similarly, studies conducted by various researchers have shown that the axial load ratio plays a decisive role in stiffness reduction, energy dissipation capacity, and ultimate strength [5,6]. Increasing axial compression has been associated with accelerated concrete crushing, reduced deformation capacity, and significant changes in cyclic response. Longitudinal reinforcement ratio and confinement provided by transverse reinforcement have also been reported to influence hysteretic stability and damage development [7]. More broadly, the behavior of SRC columns has been investigated under axial loading, bending, cyclic loading, impact, and

fire exposure [1,8–22], demonstrating the complex response of these composite members under different loading conditions.

The behavior of SRC columns depends not only on the individual mechanical properties of steel and concrete, but also on the interaction between these two materials. Load transfer at the steel–concrete interface directly affects the composite member’s stiffness, strength, deformation capacity, and failure mechanism [23]. Accurate representation of interface behavior is therefore essential for realistic numerical simulation of SRC columns.

However, in the vast majority of existing numerical studies, the interaction between steel and concrete is modeled using the assumption of a perfect bond [24]. Although this approach significantly simplifies the modeling process, it disregards the bond-slip behavior, relative displacements, and friction effects that occur at the interface. Experimental studies, however, show that shear can occur between steel and concrete, particularly under high axial load levels and large deformations, and that this can affect the load-transfer mechanism. Advances in nonlinear finite element modeling enable surface-to-surface contact, friction, and separation to be represented explicitly, thereby providing a more realistic description of steel–concrete interface behavior [10,23,25].

A review of the existing literature reveals that there are numerous studies on the cyclic behavior of SRC columns, the effects of axial loads, and finite element modeling. However, the vast majority of studies address the interaction between steel and concrete under the assumption of perfect bonding and do not directly examine the effects of interface behavior. Moreover, much of the available research addressing bond–slip behavior concerns general composite systems or other types of composite structural members rather than SRC columns subjected to cyclic loading. The scopes of the major studies in the literature are summarized in Table 1.

Table 1. Summary of previous studies related to cyclic behavior and interface modeling of SRC columns.

Study	Cyclic Loading	Axial Load Effect	Bond-Slip Behavior	Interface Friction	Numerical Modeling
Liu et al. [4]	✓	✓	✗	✗	✓
Gautham and Sahoo [5]	✓	✓	✗	✗	✓
Jiang and Bai [6]	✓	✓	✗	✗	✗
Yan et al. [7]	✓	✓	✗	✗	✗
Wang et al. [23]	✗	✗	✓	✗	✓
Ren et al. [25]	✓	✗	✓	✗	✓
Present study	✓	✓	✓	✓	✓

As shown in Table 1, while existing studies generally focus on the cyclic behavior of SRC columns and the effects of axial loads, the effects of steel–concrete interface behavior have been investigated to a limited extent. In particular, the effects of the interaction between bond conditions and the axial load ratio on cyclic behavior have not been systematically investigated. In addition, the role of interfacial friction in hysteresis behavior, stiffness reduction, and energy dissipation capacity has not yet been fully elucidated. For this reason, it is necessary to evaluate the impact of realistically modeling the steel–concrete interface on the seismic performance predictions of SRC columns.

In this study, a three-dimensional nonlinear finite element model was developed in the ABAQUS [26] environment to investigate the cyclic behavior of SRC columns. Subsequently, a comprehensive parametric study was conducted to evaluate the effects of the tie condition, axial load ratio, interface friction coefficient, and longitudinal reinforcement ratio on the behavior of SRC columns. The seismic performance of the models was evaluated in terms

of hysteresis behavior, lateral load-bearing capacity, reduction in stiffness, and energy dissipation capacity. The primary objective of this study is to highlight the limitations of the widely used assumption of perfect bonding and to quantitatively determine the importance of interface behavior in evaluating the seismic performance of SRC columns.

2. Numerical Modeling and Analysis Framework

In this study, the seismic behavior of steel-reinforced concrete (SRC) columns under cyclic lateral loads, such as those caused by earthquakes, was investigated using the finite element method. Numerical analysis was performed using ABAQUS software (version 6.12) to investigate the effect of steel-concrete interface behavior and selected design parameters on the structural performance of SRC columns.

In the parametric study, the type of bond interaction, the axial load ratio, the interfacial friction coefficient, and the longitudinal reinforcement ratio were identified as variable parameters. These parameters were selected because previous experimental and numerical studies have shown that they are the key variables determining the cyclic behavior of SRC columns [4–7,23,25]. In contrast, the concrete strength, structural steel cross-section, column geometry, transverse reinforcement arrangement, and loading protocol were kept constant across all models. The aim was thus to evaluate the individual effect of each parameter on structural behavior independently of the other variables.

The parametric analysis matrix developed for this purpose is presented in Table 2. The parametric study was designed to include three independent groups. In the first group, the combined effect of the bond condition and the axial load ratio was investigated; in the second group, the effect of the interface friction coefficient on cyclic behavior under different axial load levels was investigated. In the third group, the contribution of different longitudinal reinforcement ratios to the seismic performance of SRC columns was evaluated. Thanks to this systematic approach, the effect of each parameter on lateral load-carrying capacity, changes in stiffness, and energy dissipation capacity has been demonstrated separately.

In this study, three different axial load ratios were considered to represent low ($n = 0.10$), medium ($n = 0.50$), and high ($n = 0.85$) axial load levels, which are commonly used in practical SRC column applications and previous cyclic loading studies [2–5,25]. The axial load ratio is defined as the ratio of the applied axial load to the axial load-carrying capacity of the SRC column ($n = N/(N_0)$). The axial load-carrying capacity (N_0) of the SRC column was calculated as follows using Equation (1).

$$N_0 = 0.85f_c(A_c - A_s - A_{sr}) + f_yA_s + f_{yr}A_{sr} \quad (1)$$

Here, f_c represents the compressive strength of concrete, A_c represents the total cross-sectional area of concrete, A_s represents the cross-sectional area of steel reinforcement, A_{sr} represents the cross-sectional area of structural steel, f_y represents the yield strength of steel reinforcement and f_{yr} represents the yield strength of structural steel.

Equation (1) was adopted from the axial load-carrying capacity formulation given in TS 500 [27]. The origin of the coefficient 0.85 applied to the concrete contribution is experimental. As explained by Ersoy and Özcebe [28], experimental observations on axially loaded columns showed that the compressive strength of concrete in a column is, on average, approximately 85% of the corresponding cylinder compressive strength. Accordingly, the concrete contribution to the axial load-carrying capacity is represented by $(0.85f_cA_c)$. The concrete used in the present study is within the normal-strength concrete range for which this conventional formulation is employed. Since the cited formulation does not specify a separate strength-dependent value of this coefficient within this range, a constant value of 0.85 was adopted in the present calculations.

Table 2. Parametric analysis matrix for the current study.

Group Number	Objective of the Parametric Study	Model Name	Bond Condition	Axial Load Ratio ($n = \frac{N}{N_0}$)	Axial Load N (Newton)	Coefficient of Friction (μ)	Steel Ratio	Section Rebar
1	Effect of bond condition and axial load ratio	SRC-1	Perfect	0.1	361,868	–	2.0%	8Ø17
		SRC-2	Perfect	0.5	1,809,338	–	2.0%	
		SRC-3	Perfect	0.85	3,075,875	–	2.0%	
		SRC-4	Friction	0.1	361,868	0.4	2.0%	
		SRC-5	Friction	0.5	1,809,338	0.4	2.0%	
		SRC-6	Friction	0.85	3,075,875	0.4	2.0%	
2	Effect of interface friction coefficient	SRC-7	Friction	0.1	361,868	0.0	2.0%	8Ø17
		SRC-8	Friction	0.85	3,075,875	0.0	2.0%	
		SRC-10	Friction	0.1	361,868	0.2	2.0%	
		SRC-11	Friction	0.1	361,868	0.6	2.0%	
		SRC-12	Friction	0.85	3,075,875	0.2	2.0%	
		SRC-13	Friction	0.85	3,075,875	0.6	2.0%	
3	Effect of longitudinal reinforcement ratio	SRC-14	Perfect	0.1	337,748	–	1.5%	8Ø14.7
		SRC-15	Perfect	0.1	412,151	–	3.0%	8Ø21
		SRC-16	Perfect	0.85	2,870,861	–	1.5%	8Ø14.7
		SRC-17	Perfect	0.85	3,503,281	–	3.0%	8Ø21

Equation (1) also considers the contributions of the reinforcing steel and structural steel separately through their respective cross-sectional areas and yield strengths. The yield-to-tensile strength ratios (f_y/f_u) of these steel components are not included as independent parameters in this formulation. In the present study, Equation (1) is used specifically to determine the reference axial load-carrying capacity (N_0), required to define the applied axial load ratio ($n = N/N_0$), rather than to characterize the post-yield or ultimate tensile behavior of the reinforcing steel and structural steel. Therefore, the different yield-to-tensile strength ratios of the two steel components are outside the scope of this reference axial-capacity calculation.

Previous experimental and numerical studies have shown that the axial load ratio significantly affects the cyclic behavior, stiffness reduction, ductility, and failure mode of SRC and composite columns [4–6,29]. While low axial load levels generally promote ductile bending behavior, high axial load levels accelerate the crushing of concrete and lead to more brittle responses. For this reason, the axial load ratios selected in the present study were determined to represent the change in performance of SRC columns from low axial load levels to the behavior region where the effect of high pressure predominates. In this study, axial load ratios greater than 0.85 were not included in the parametric study because such loading conditions generally lead to early pressure failure and limited ductility, and therefore are less representative of earthquake-resistant SRC column design.

In this study, longitudinal reinforcement ratios of 1.5%, 2.0%, and 3.0% were used to investigate the effect of the steel reinforcement ratio on the seismic behavior of SRC columns. For all models, the structural steel cross-sectional properties, concrete strength, and other geometric properties were kept constant; only the longitudinal reinforcement ratio was changed. For this reason, the term “reinforcement ratio” refers to the ratio of the longitudinal reinforcement area (A_s) to the column’s cross-sectional area (A_c) (Equation (2)).

$$p_s = \frac{A_s}{A_c} \quad (2)$$

Previous experimental and numerical studies have shown that the longitudinal reinforcement ratio significantly affects the strength, stiffness, ductility, confinement behavior, and energy dissipation capacity of composite and SRC columns [4,7]. Increasing the reinforcement ratio generally increases the lateral load-carrying capacity and delays the onset of stiffness loss; however, excessively high reinforcement ratios can alter the damage mechanisms and reduce the deformation capacity under cyclic loading. In addition, studies conducted on composite columns subjected to seismic loads have shown that reinforcement ratios ranging from approximately 1.0% to 3.0% provide practical and representative ranges for evaluating ductility and energy dissipation performance [4]. For this reason, it has been assumed that the reinforcement ratios selected in the parametric study are sufficient to represent the effect of reinforcement on the behavior of SRC columns.

According to the established analytical framework, the study consists of three main phases. In the first stage, the geometric properties and reinforcement arrangement of the SRC column were defined. Using the geometry created in the second stage, a nonlinear finite element model was developed, and material models and steel-concrete interface interactions were defined. In the third stage, the developed model was validated using experimental results, and parametric analyses were then conducted. The detailed analysis process followed in this study is summarized in the workflow diagram shown in Figure 2.

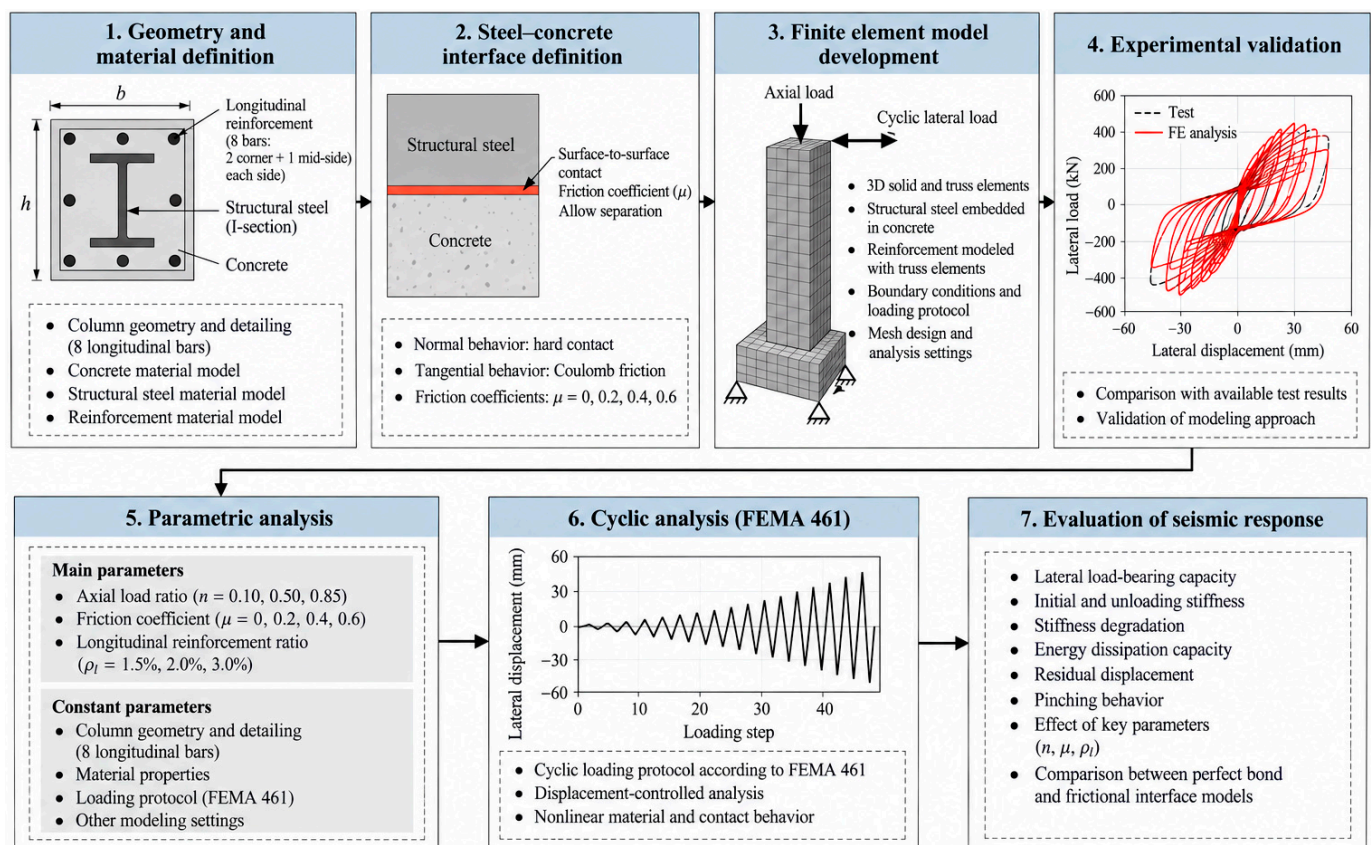


Figure 2. Workflow of the numerical procedure adopted in the present study.

2.1. Geometry and Structural Configuration

The SRC column model used in this study was developed to represent the typical column geometry commonly used in mid-rise and high-rise steel-concrete composite structures. To ensure that the results obtained from the parametric analyses were influenced only by the selected variables, the column geometry was kept constant in all analyses, and

only the bond condition, axial load ratio, interface friction coefficient, and longitudinal reinforcement ratio were varied.

The cross-section of the SRC column element considered in this study was selected as 30×30 cm; the use of a full-scale column eliminates uncertainties related to geometric scaling and ensures a more realistic representation of the structural behavior under cyclic loading. The aim is for the numerical results obtained through this approach to represent real-world engineering applications, independent of scale effects. Concrete of strength class C30/37 (EN 1992-1-1 [30] notation, corresponding to a characteristic cylinder compressive strength of $f_{ck} = 30$ MPa and cube strength of 37 MPa) was used in the column. An ISMB125 profile was used as a structural steel member embedded within the column cross-section. The ISMB125 profile was selected to provide an appropriate balance between axial load capacity, bending stiffness, and composite behavior within a $300 \text{ mm} \times 300 \text{ mm}$ cross-section.

Longitudinal reinforcements are symmetrically placed within the concrete cross-section and contribute to both the axial load-carrying capacity and the flexural strength. Transverse reinforcement, on the other hand, helps to wrap the concrete core, thereby delaying local buckling, limiting the lateral expansion of the concrete, and enhancing ductile behavior under cyclic loading. The reinforcement layout and structural steel configuration used in the numerical model are shown in Figure 3.

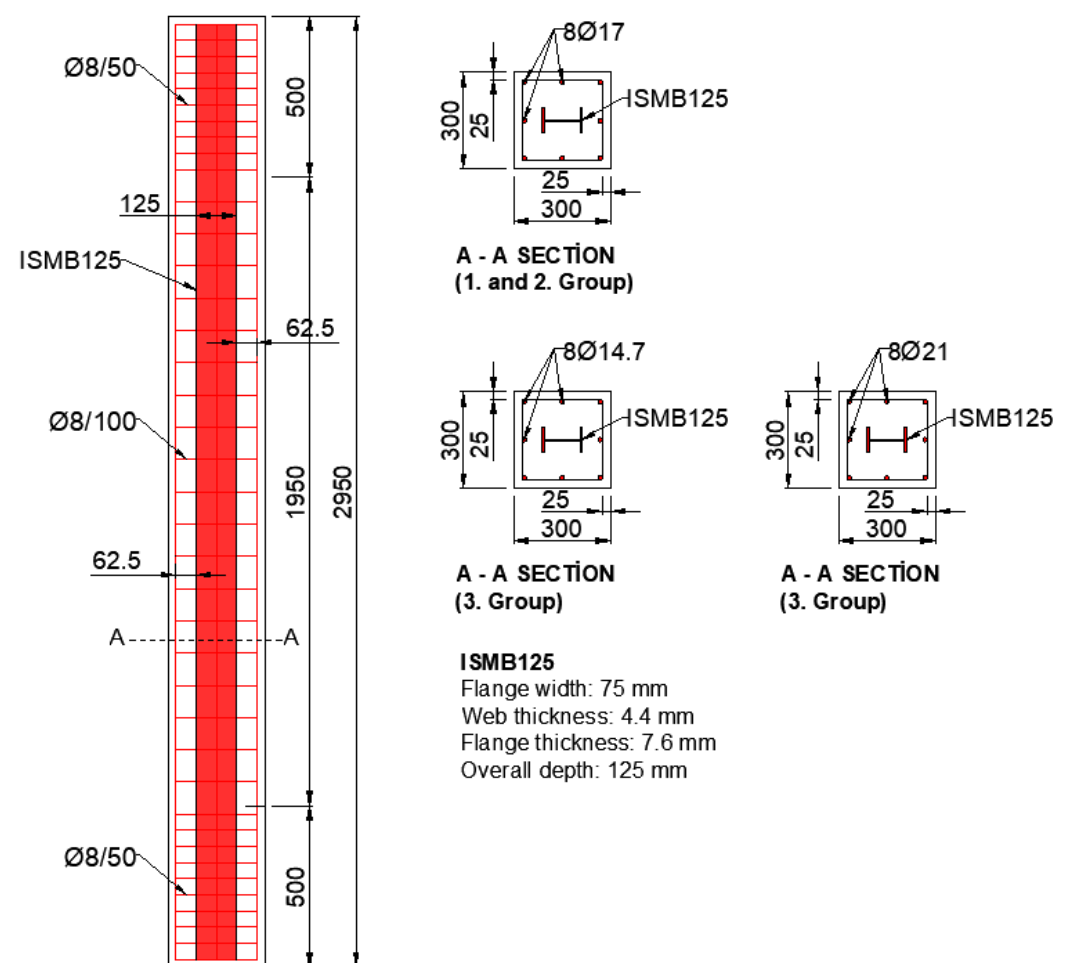


Figure 3. Geometry, cross-sectional details, reinforcement placement, and embedded ISMB125 structural steel profile used in the numerical model (all dimensions are in mm).

When creating the geometric model, a cross-section layout was chosen that would allow for the proper generation of the finite element mesh and the accurate definition of the steel–concrete interface. This ensures that the changes in behavior observed during parametric analyses are attributable not to geometric differences but directly to the parameters under investigation. This approach allows for a reliable comparison of the effects of different bonding conditions and interface properties on the cyclic behavior of SRC columns.

2.2. Finite Element Modeling of the SRC Column Under Cyclic Lateral Loading

In this study, a three-dimensional finite element model was developed using ABAQUS software to investigate the nonlinear behavior of SRC columns under cyclic loading. The model was developed to account for the cracking and crushing behavior of concrete, the plastic deformation of steel, and the interaction mechanisms at the steel–concrete interface. In addition, geometric nonlinearity effects (NLGEOM = ON) were enabled throughout the analyses to account for the effects of large displacements on structural behavior.

The concrete body forming the SRC column was modeled using three-dimensional solid elements with reduced integration and eight nodal points (C3D8R) (Figure 4a). This element type was chosen because it is compatible with concrete damage models, reduces computational costs, and ensures numerical stability in nonlinear analyses. The structural steel profile was also modeled using three-dimensional solid elements. The use of solid elements allows for a more realistic representation of the stress distributions and contact behavior at the steel–concrete interface. Longitudinal and transverse reinforcements were modeled using two-node truss elements (T3D2) that carry only axial forces. This approach accurately represents the mechanical behavior of reinforcing steel while also improving computational efficiency (Figure 4b).

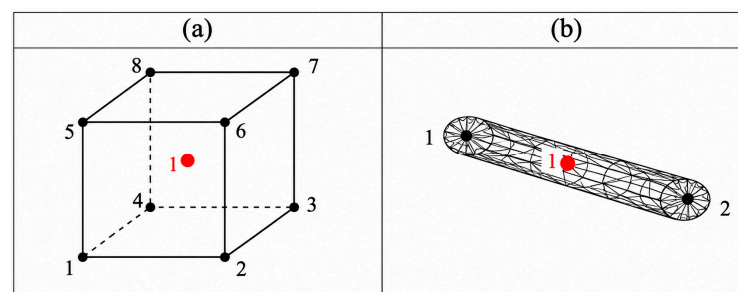


Figure 4. (a) C3D8R element; (b) T3D2 element.

When creating the finite element mesh, a structured mesh was chosen for the concrete and structural steel components, and an element size of approximately 50 mm was used in the analyses. This mesh configuration ensures a uniform stress distribution while also striking an appropriate balance between computation time and analysis accuracy. By using the same network strategy in all parametric analyses, it was ensured that the observed differences in behavior were attributable solely to the parameters under study.

The finite element discretization adopted in the present study is illustrated in Figure 5. The figure presents the concrete body, embedded ISMB125 structural steel profile, reinforcement cage, and the assembled numerical model used throughout the parametric analyses.

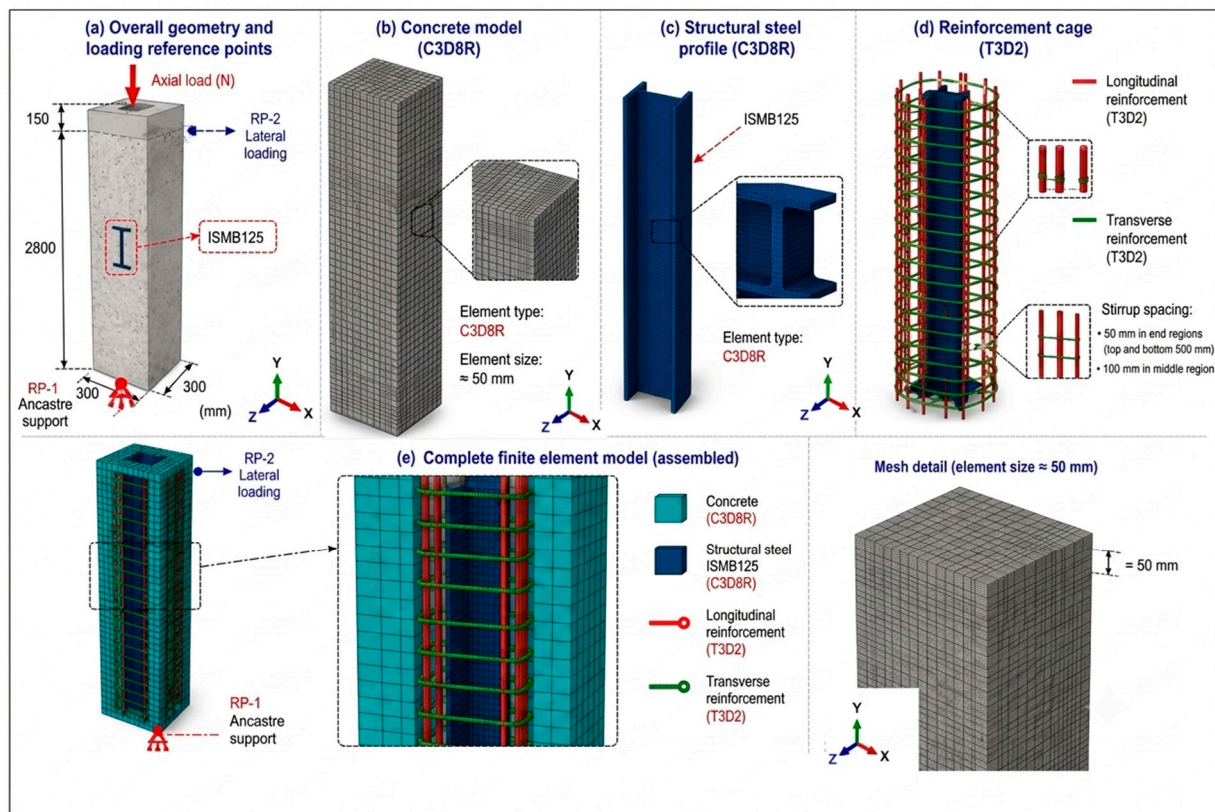


Figure 5. Three-dimensional finite element idealization of the SRC column showing (a) concrete body, (b) embedded ISMB125 structural steel profile, (c) reinforcement cage modeled using T3D2 elements, (d) assembled finite element model, and (e) finite element discretization adopted in the numerical analyses.

2.2.1. Material Models

Concrete Material Model

The nonlinear behavior of concrete was simulated using the Concrete Damage Plasticity (CDP) model available in ABAQUS. The CDP model is a formulation that can represent the inelastic behavior of concrete by accounting for stiffness degradation associated with tensile cracking and compressive failure. Due to its ability to reproduce the development of irreversible damage under monotonic and cyclic loading, the CDP model has been widely adopted in the nonlinear finite element analysis of reinforced concrete and steel-concrete composite members.

In the present study, the CDP model was chosen because the SRC columns under examination were subjected to cyclic lateral loads that caused the concrete to experience repeated cracking, crushing, and loss of stiffness. Unlike traditional elastic-plastic material models, the CDP formulation captures the gradual degradation of concrete's mechanical properties through independent tensile and compressive damage variables, thereby providing a more realistic simulation of the cyclic response of composite columns.

The stress–strain relationship of concrete under compression and tension has been defined in accordance with the recommendations of the CEB/FIB Model Code [31]. The relevant structural relationships between compression and tension adopted in the numerical analyses are shown in Figure 6. These constitutive models allow for the inclusion of the gradual reduction in concrete stiffness following cracking and crushing in the nonlinear analysis.

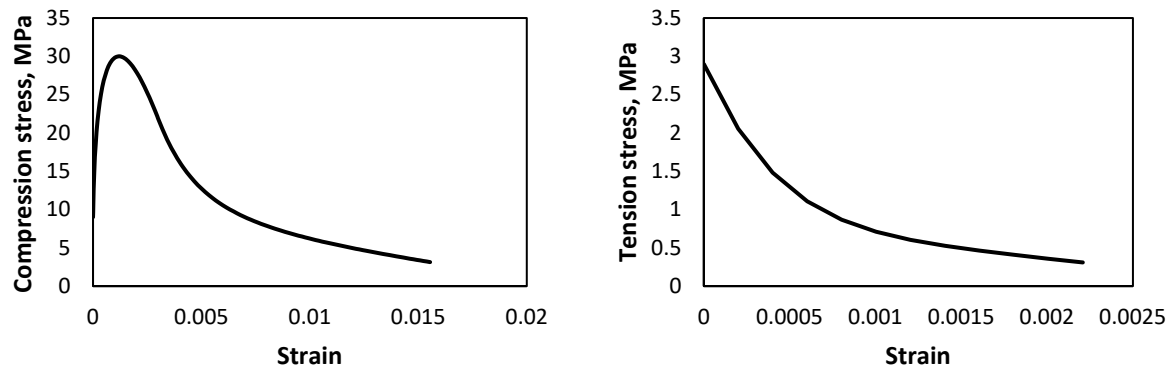


Figure 6. Uniaxial stress–strain relationships of concrete used in the Concrete Damage Plasticity (CDP) model.

The damage development of concrete has been demonstrated through compressive and tensile damage variables, which describe the progressive deterioration of elastic stiffness under increasing inelastic deformation. The adopted damage progression curves are shown in Figure 7 and have been directly implemented within the CDP material model to accurately simulate the degradation of concrete under cyclic loading.

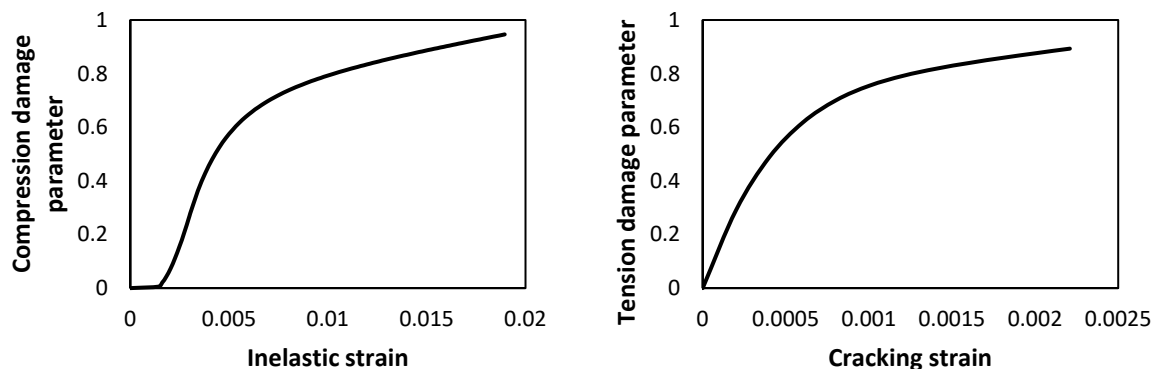


Figure 7. Stiffness reduction parameters for concrete based on compressive and tensile damage, as adopted in the CDP material model.

Structural Steel and Reinforcing Steel Material Models

Structural steel embedded in concrete (ISMB125) and reinforcing steel were modeled using an elastic-plastic constitutive relationship with isotropic stress hardening available in ABAQUS. This material model effectively simulates the nonlinear response of steel by taking into account both the elastic behavior prior to yielding and the plastic deformation and strain hardening that occur after yielding. This approach has been widely adopted in nonlinear finite element analyses of reinforced concrete and steel-concrete composite structures due to its ability to realistically reproduce the cyclic response of steel members.

Structural steel and reinforcing steel are assumed to be homogeneous and isotropic, with an elastic modulus of 200 GPa and a Poisson's ratio of 0.3. Structural steel and reinforcing steel were assumed to be homogeneous and isotropic, with an elastic modulus of 200 GPa and a Poisson's ratio of 0.3. The yield strengths of the structural steel and reinforcing steel were taken as 250 MPa and 550 MPa, respectively, based on the experimentally determined material properties reported by Gautham and Sahoo [5]. In particular, the reinforcing-steel yield strength of 550 MPa was not selected as a nominal reinforcement grade associated with the C30/37 concrete adopted in the parametric models; rather, it was retained from the experimental material data used as the basis for the finite element model. Therefore, the value represents the measured yield strength of the reinforcing steel rather than a nominal design-grade specification. The source of this value is explicitly stated

here to distinguish the experimentally based material property from commonly specified nominal reinforcement grades.

The stress–strain data used to define the constitutive behavior of both the embedded structural steel and reinforcing steel were also adopted from the experimental material data reported by Gautham and Sahoo [5]. Accordingly, the uniaxial stress–strain relationships implemented in the finite element model and presented in Figure 8 are based on the material data reported in the reference experimental study. The same material definitions were retained throughout the parametric analyses to maintain consistency with the validated reference model.

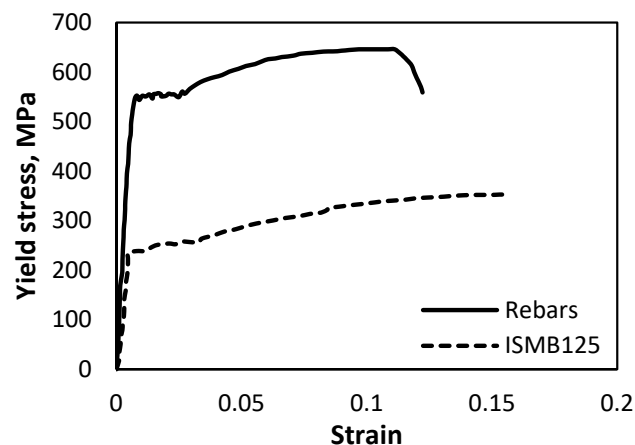


Figure 8. Uniaxial stress–strain relationships adopted for the embedded structural steel and reinforcing steel based on the experimental material data reported by Gautham and Sahoo [5].

The selected structural stress–strain relationships make it possible to represent the yield and stress-hardening behavior of both embedded structural steel and reinforcing steel while maintaining numerical stability throughout nonlinear cyclic analyses. As a result, the interaction between concrete damage and steel plasticity can be realistically captured throughout the entire loading history.

Concrete Damage Plasticity (CDP) Parameters

The ABAQUS [26] software uses various plasticity parameters for the CDP material model, including the dilation angle (ψ), eccentricity (e), viscosity (μ), and the ratio of the yield stress in biaxial tension to that in uniaxial tension (f_{bo}/f_{co}). These parameters govern the nonlinear behavior of concrete following cracking and crushing and, therefore, play an important role in accurately predicting the cyclic behavior of reinforced concrete and steel-concrete composite members. The CDP parameters used in this study were selected based on recommendations reported in previous experimental and numerical studies [7,32–37] involving reinforced concrete members subjected to cyclic loading. These parameters have been widely used in ABAQUS-based nonlinear analyses and have demonstrated a satisfactory ability to reproduce the stiffness degradation, damage accumulation, and hysteretic response of concrete members under cyclic loading. The CDP parameters used in numerical analyses are summarized in Table 3. This selected set of parameters realistically represents the inelastic behavior of concrete while providing a numerically stable solution to convergence problems. This ensures that the finite element model developed captures the gradual deterioration of the SRC columns throughout the cyclic loading history.

Table 3. Plasticity parameters used in the present study.

Parameter	Value	Physical Meaning
Dilation angle (ψ)	36°	Controls volumetric expansion during plastic flow
Eccentricity (e)	0.10	Defines the plastic potential function
f_{b0} / f_{c0}	1.16	Ratio of biaxial to uniaxial compressive strength
K_c	0.667	Shape factor of the yield surface
Viscosity parameter (μ)	0.0005	Improves numerical convergence

2.2.2. Steel—Concrete Interface Modeling

The seismic performance of steel-reinforced concrete (SRC) columns under cyclic loading is determined not only by the individual mechanical properties of the concrete and structural steel, but also by the interaction mechanism that develops at the steel-concrete interface. Under cyclic lateral loads, stress transfer between structural steel embedded in concrete and the surrounding concrete is achieved through various combinations of adhesive bonding, mechanical interlock, and frictional resistance. As damage accumulates at the structural steel-concrete interface over the course of loading, separation and relative slip conditions occur. As a result of this situation, changes occur in the cyclic behavior of SRC columns due to a decrease in stiffness.

Despite this effect of structural steel-concrete interface behavior on SRC columns, many studies in the literature have adopted the assumption of perfect bond, in which the deformation between steel and concrete is assumed to be fully synchronized throughout the analysis. Although this assumption simplifies modeling in numerical analyses and reduces analysis time, it neglects interface effects such as separation and slipping that occur in actual behavior under cyclic loading. For this reason, the interface model adopted can significantly affect the predicted stiffness, ductility, and energy dissipation capacity of SRC columns.

In this study, two different approaches to the interaction at the structural steel-concrete interface are discussed. One of these approaches involves perfect bond interaction, while the other involves frictional bond interaction. These models were evaluated under the same geometric configuration, material properties, and loading conditions in order to quantitatively determine the isolated effect of interface modeling.

Perfect Bond Model

The Perfect Bond Model represents the traditional interface assumption used for SRC elements in many finite element analyses. Under this assumption, no relative displacement is allowed between the structural steel embedded in the concrete and the concrete surrounding it. This indicates that the deformations are fully consistent throughout the loading history. In the perfect bond interaction type, normal and shear stresses are fully transferred across the interface without any shear or separation.

In the present study, the perfect bond condition was implemented in ABAQUS using a Tie Constraint, which rigidly couples the structural steel surface to the surrounding concrete. This modeling approach assumes that the composite action between steel and concrete remains fully effective throughout cyclic loading.

Although the assumption of a perfect bond interaction significantly simplifies numerical analysis models, it neglects the interface distortion caused by cyclic loading. As a result, the estimated stiffness, load-carrying capacity, and energy dissipation capacities may be overestimated when compared to the actual structural response. However, in the present

study, the perfect bond interaction model was adopted as the reference configuration against which the frictional bond interaction model was evaluated.

Frictional Bond Model

To capture more realistic behavior at the interface between structural steel and concrete, the assumption of a frictional bond interaction type is used. In this approach, unlike the assumption of a perfect bond interaction, pressure load transfer via surface contact is maintained, while relative slip at the interface between structural steel and concrete is allowed. As a result, the interface behavior evolves based on the applied loading history and provides a realistic simulation of seismic performance under cyclic loading.

In this study, the surface-to-surface contact interaction between the outer surface of structural steel embedded in concrete and the corresponding concrete surface has been defined. Normal contact behavior was modeled using the Hard Contact formulation, which prevents penetration under compressive load but allows separation when tensile stress occurs. The tangential behavior was represented using the Penalty Friction formulation based on the classical Coulomb friction law.

In this study, four different friction coefficients ($\mu = 0.0, 0.20, 0.40$, and 0.60) were considered to evaluate the effect of interface friction on the seismic behavior of SRC columns under cyclic loading. These values represent the different levels of friction resistance that may develop following the failure of the adhesive bond between structural steel and concrete. By varying the friction coefficient while keeping all other modeling parameters constant, the isolated effect of interface friction on the hysteresis response, stiffness degradation, and energy dissipation capacity of SRC columns was systematically investigated.

Compared to the assumption of a perfect bond interaction, the assumption of a frictional bond interaction allows for the occurrence of local shear and separation effects, which are frequently observed in experimental studies of SRC columns subjected to cyclic loading. For this reason, this modeling strategy simulates the mechanical interaction between steel and concrete more realistically throughout the loading process.

In the numerical analysis of the seismic performance of SRC columns, although more advanced interface formulations such as the cohesive bond interaction model can represent detachment and interface slip damage, their application generally requires a large number of material parameters that must be calibrated using specialized experimental bond-slip tests. Since such experimental data were not available within the scope of this study, the steel-concrete interaction was modeled using perfect bond and friction-bond models. These two approaches allow for the investigation of the effects of interface slipping and friction resistance on the cyclic behavior of SRC columns, while maintaining a physically meaningful and numerically robust modeling framework.

2.3. Static Analysis Under Cyclic Lateral Loading

The seismic performance of SRC columns was evaluated using nonlinear semi-static analyses under displacement-controlled cyclic lateral loading. Displacement control has been adopted because it captures the column's entire nonlinear response, including a decrease in stiffness, yielding, and post-peak behavior more reliably than load-controlled analyses, particularly under severe cyclic loading conditions.

The cyclic loading history has been defined in accordance with the recommendations of FEMA 461 [38], which is widely used for the seismic evaluation of reinforced concrete and composite structural members. The FEMA 461 loading protocol allows for the systematic evaluation of damage accumulation, stiffness degradation, and energy dissipation characteristics of structural members by applying progressively increasing displacement amplitudes [39].

The loading protocol adopted in this study is shown in Figure 9. The loading protocol starts with a small displacement amplitude corresponding to the column's elastic response and increases by approximately 40% at each subsequent loading stage. Symmetrical displacement cycles were applied in both the positive and negative loading directions until the target deviation ratio was reached.

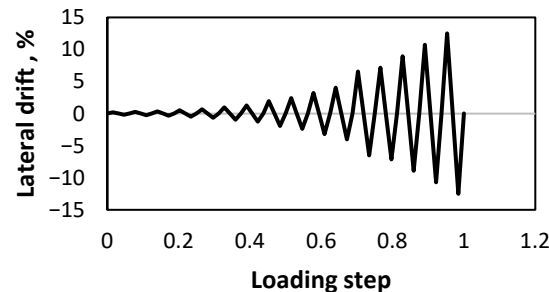


Figure 9. Displacement-controlled cyclic loading protocol adopted according to FEMA 461 [38].

In the analyses, the NLGEOM option has been enabled to account for geometric non-linearities resulting from large displacements. All analyses were performed using the Static, General analysis procedure. While the specified displacement history was applied to the RP-2 reference point, located 150 mm below the top surface of the column, via an amplitude function corresponding to the cyclic loading protocol shown in Figure 8, a constant axial load was maintained throughout the entire loading history. This loading strategy allows for the evaluation of the hysteretic response of SRC columns under gradually increasing deformation demands that represent seismic loading.

2.4. Calculation of Seismic Performance Indicators

In this study, the seismic behavior of SRC columns under cyclic lateral loading was evaluated using three general performance indicators derived from lateral load–lateral displacement curves. These seismic performance indicators are initial stiffness, normalized stiffness reduction ratio, and energy dissipation capacity.

The initial stiffness of the SRC columns was evaluated using the secant stiffness approach—which is commonly used in studies applying cyclic lateral loads to assess the reduction in structural stiffness during lateral loading cycles. Secant stiffness was calculated for each loading cycle as the ratio of the maximum lateral load to the corresponding horizontal displacement, using Equation (3) (Figure 10).

$$K_i = \frac{F_i}{\Delta_i} \quad (3)$$

where F_i is the lateral load and Δ_i is the corresponding lateral displacement at the peak of the i^{th} loading cycle. The initial stiffness obtained from the first loading cycle is denoted by K_0 .

To evaluate the loss of structural stiffness during cyclic loading, the normalized stiffness reduction ratio K_i/K_0 was calculated (Figure 10). The normalized stiffness reduction ratio provides a suitable measure for comparing stiffness degradation among different numerical models, regardless of their initial stiffness values.

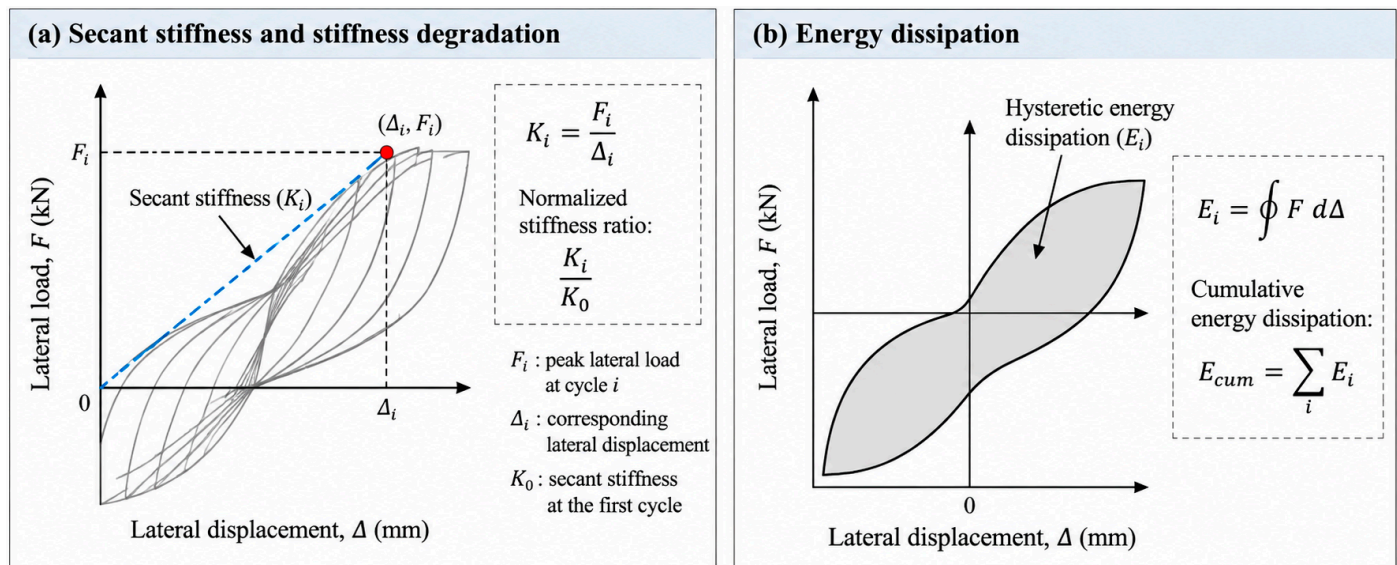


Figure 10. Definition of the seismic performance indices adopted in this study: (a) secant stiffness and normalized stiffness ratio and (b) hysteretic energy dissipation calculated from the enclosed load–displacement loop area.

Where K_i represents the secant stiffness at the i^{th} loading cycle and K_0 is the initial stiffness obtained from the first loading cycle.

Energy dissipation is one of the most important indicators of seismic performance, as it reflects a structural element's ability to absorb and dissipate earthquake-induced energy through inelastic deformation.

In this study, the energy dissipation capacity of SRC columns was evaluated using the area enclosed by the hysteresis loops obtained from cyclic lateral load analyses. Accordingly, for each loading cycle, the dissipated energy was calculated using Equation (4) as the area enclosed within the load–displacement hysteresis loop. Total energy dissipation was calculated by summing the energy values corresponding to all loading cycles using Equation (5) (Figure 10).

$$E_i = \int F d\Delta \quad (4)$$

$$E_{cum} = \sum_{i=1}^n E_i \quad (5)$$

where P denotes the lateral load and Δ is the corresponding lateral displacement. The enclosed hysteretic area represents the mechanical energy dissipated through irreversible deformation during each loading cycle (E_i), while the cumulative energy dissipation corresponds to the sum of all individual hysteresis loop areas (E_{cum}).

It exhibits higher cumulative energy consumption, improved seismic performance, and greater resistance to cyclic damage.

To further characterize the influence of interface friction on the hysteretic response, residual displacement, unloading stiffness, and pinching behavior were additionally evaluated at the 8% drift cycle. The residual displacement was determined from the zero-force intercept of the unloading branch by linear interpolation, while the unloading stiffness was calculated as the secant slope between the peak-response point and the corresponding zero-force intercept. Pinching behavior was quantified using a pinching force ratio, defined as the absolute lateral force at zero displacement normalized by the corresponding peak lateral force of the cycle. For each model, the reported values represent the average of the positive and negative loading directions.

2.5. Numerical Model Validation

Before the parametric study, the finite element modeling strategy to be used in the present study was validated using the experimental results reported by Gautham and Sahoo [5]. The objective here is not to reproduce the SRC column geometry adopted in the existing parametric study, but rather to evaluate the ability of the developed numerical modeling approach, the selected structural material models, the boundary conditions, and the nonlinear solution strategy to accurately simulate the seismic behavior of steel-reinforced concrete (SRC) columns under cyclic lateral loading.

Among the SRC columns tested by Gautham and Sahoo [5], the S1 specimen was selected for the validation study because comprehensive information is available regarding its geometry, reinforcement detailing, material properties, loading setup, and boundary conditions. All of this information enabled the S1 test specimen to be modeled and analyzed in ABAQUS with a high degree of accuracy. The cross-sectional dimensions and reinforcement details of the reference specimen are shown in Figure 11.

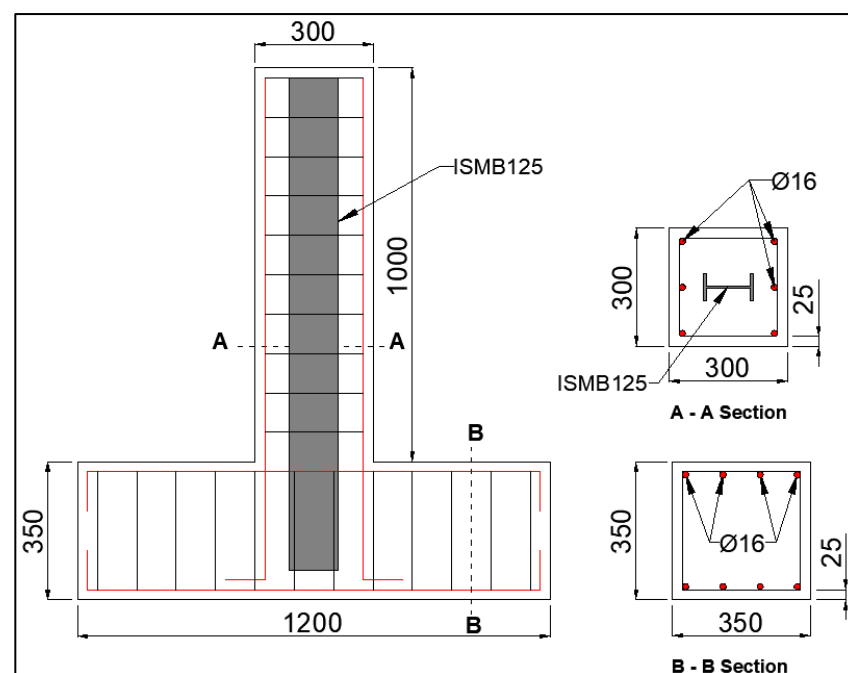


Figure 11. Cross-sectional properties and reinforcement arrangement of the S1 column element considered in the reference study [5].

The loading configuration and boundary conditions adopted in the experimental program conducted by Gautham and Sahoo [5] were simulated as realistically as possible in the numerical model. As shown in Figure 11, the S1 experimental column was subjected to a controlled displacement under a constant axial load while being exposed to a cyclic lateral load; the same boundary conditions were adopted in the finite element model.

In the experimental study, displacement-controlled cyclic lateral loading under a constant axial load was employed, and the same boundary conditions were adopted in the finite element model, as shown in Figure 12.

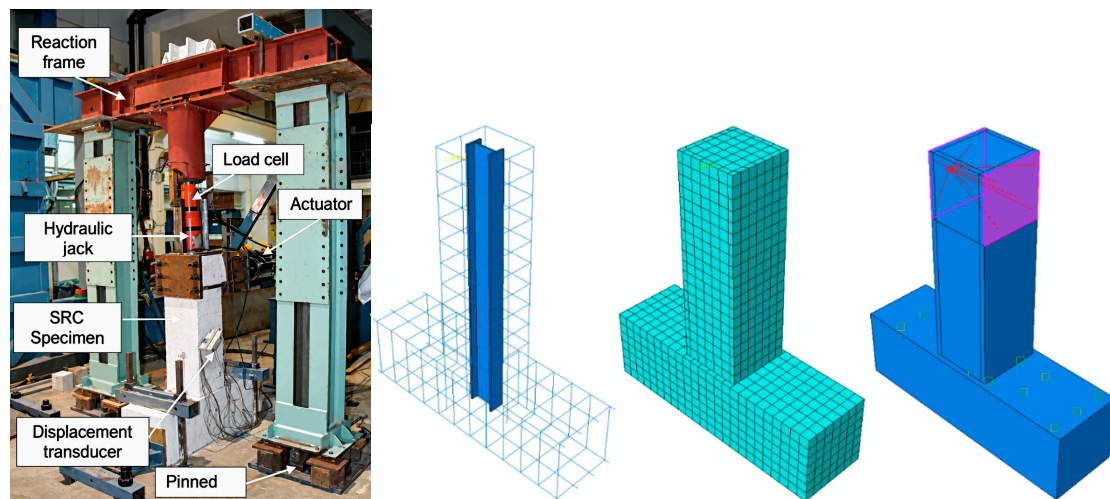


Figure 12. Loading conditions and boundary conditions in experimental and numerical analyses [5].

Although the reference S1 specimen was experimentally tested under cyclic lateral loading, the validation analysis in the present study was performed using monotonic displacement-controlled lateral loading. The purpose of this validation was to assess the capability of the finite element model to reproduce the principal global response characteristics, particularly the initial stiffness, lateral load-carrying capacity, and overall load–displacement envelope, rather than to reproduce cycle-dependent phenomena such as unloading–reloading stiffness degradation or cumulative cyclic damage. The use of monotonic numerical analyses for evaluating the global response and full-range load–deformation behavior of steel–concrete composite columns has also been adopted in previous numerical studies [40,41]. Accordingly, the present validation should be interpreted as a validation of the global response characteristics of the numerical model rather than a direct validation of its complete cyclic hysteretic response.

Following the numerical validation analysis, the lateral load–horizontal displacement relationship and the stiffness–horizontal displacement relationship obtained from the finite element model were compared with the relevant experimental results. The results are shown in Figure 13, while the main seismic performance parameters are summarized in Table 4.

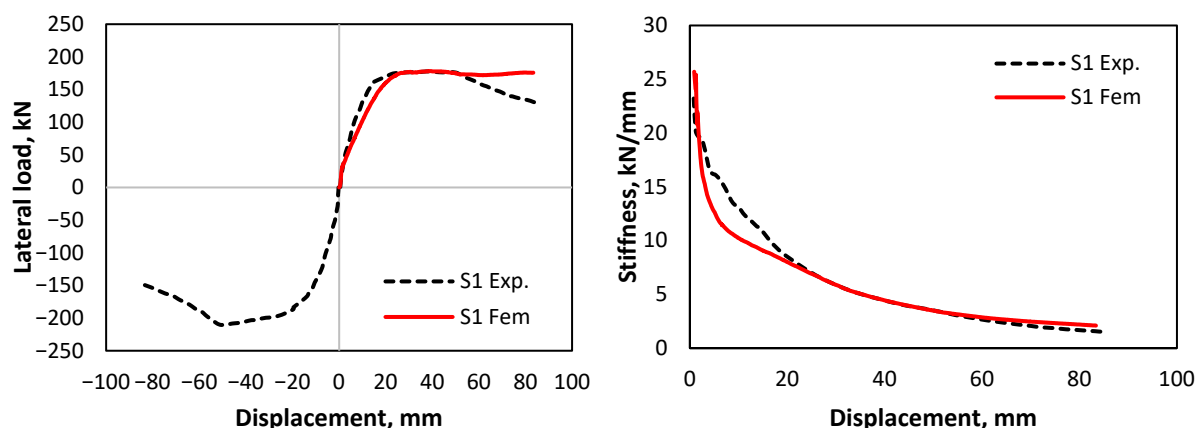


Figure 13. Comparison of experimental and numerical results for the lateral load–horizontal displacement and initial stiffness–horizontal displacement relationships for Column S1 [5].

Table 4. Comparison of basic seismic response parameters obtained from experimental and numerical analyses.

Parameter	Exp.	Fem (ABAQUS)	Error (%)
Peak lateral load (kN)	≈177.72	≈178.11	~0.22%
Displacement at peak (mm)	≈37.49	≈39.34	~4.7%
Initial stiffness (kN/mm)	≈23.22	≈25	~7.67%

As shown in Table 4, the finite element model predicts the lateral load-bearing capacity with an error of only 0.22%, demonstrating excellent agreement with experimental observations regarding lateral loads. While the displacement corresponding to the largest lateral load differed by approximately 4.7%, the initial stiffness was overestimated by 7.67%. The slightly higher initial stiffness predicted by the numerical model can be attributed primarily to the idealized modeling assumptions adopted in the analysis. In particular, the assumption of perfect bond between the embedded structural steel and the surrounding concrete neglects local interface slippage and bond failure that may occur during testing. Furthermore, the monotonic loading procedure adopted in the numerical validation does not reproduce the decrease in stiffness caused by repeated cyclic loading. While these simplifications result in a slightly stiffer structural response compared to experimental observations, they provide an excellent match in terms of peak lateral load. In conclusion, the mechanical analysis of the verified SRC column element provides a solid foundation for the expansion of SRC column systems.

3. Findings and Discussion

As a result of this study, performance indicators of the SRC column—such as lateral load-carrying capacity, stiffness, and energy dissipation capacity—were examined under cyclic lateral load conditions as a function of parameters including the axial load ratio, reinforcement ratio, the type of bond interaction between the concrete and reinforcement, and the friction coefficient.

3.1. Effect of Bond Condition and Axial Load Ratio on Seismic Performance of SRC Columns

3.1.1. Hysteretic Behavior

Figure 14 shows the lateral load–lateral drift for all numerical analyses. It should be noted that the analyses were terminated upon reaching the target drift ratio prescribed by the FEMA 461 [38] loading protocol, rather than upon a predefined strength-degradation criterion (e.g., a percentage drop from the peak load). Consequently, in the final loading cycle of several models, a secondary increase in lateral load capacity can be observed following the initial post-peak decline (Figure 14). This behavior is attributed to a shift in load-carrying mechanism: as progressive crushing reduces the concrete's contribution to lateral resistance, an increasing proportion of the load is transferred to the embedded steel section, which is modeled with isotropic strain hardening. As the steel enters its hardening range at large plastic strains, its resistance continues to increase even as the concrete's contribution diminishes, producing the observed net rise at the final cycle. This reflects a genuine transition in composite action toward steel-dominated behavior rather than a numerical artifact, and does not indicate that the true post-peak degradation trend would reverse under continued loading.

Figure 15 shows the envelope curves obtained from the hysteresis curves of SRC columns subjected to cyclic lateral loads at different types of bond interactions and different axial load ratios. Modeling the structural steel within the SRC column and the surrounding concrete using different interface modeling approaches results in a significant difference in cyclic behavior. It is observed that SRC columns modeled under the assumption of a perfect bond condition exhibit more stable cyclic responses and a higher lateral load capacity compared to those modeled under the assumption of a frictional bond condition.

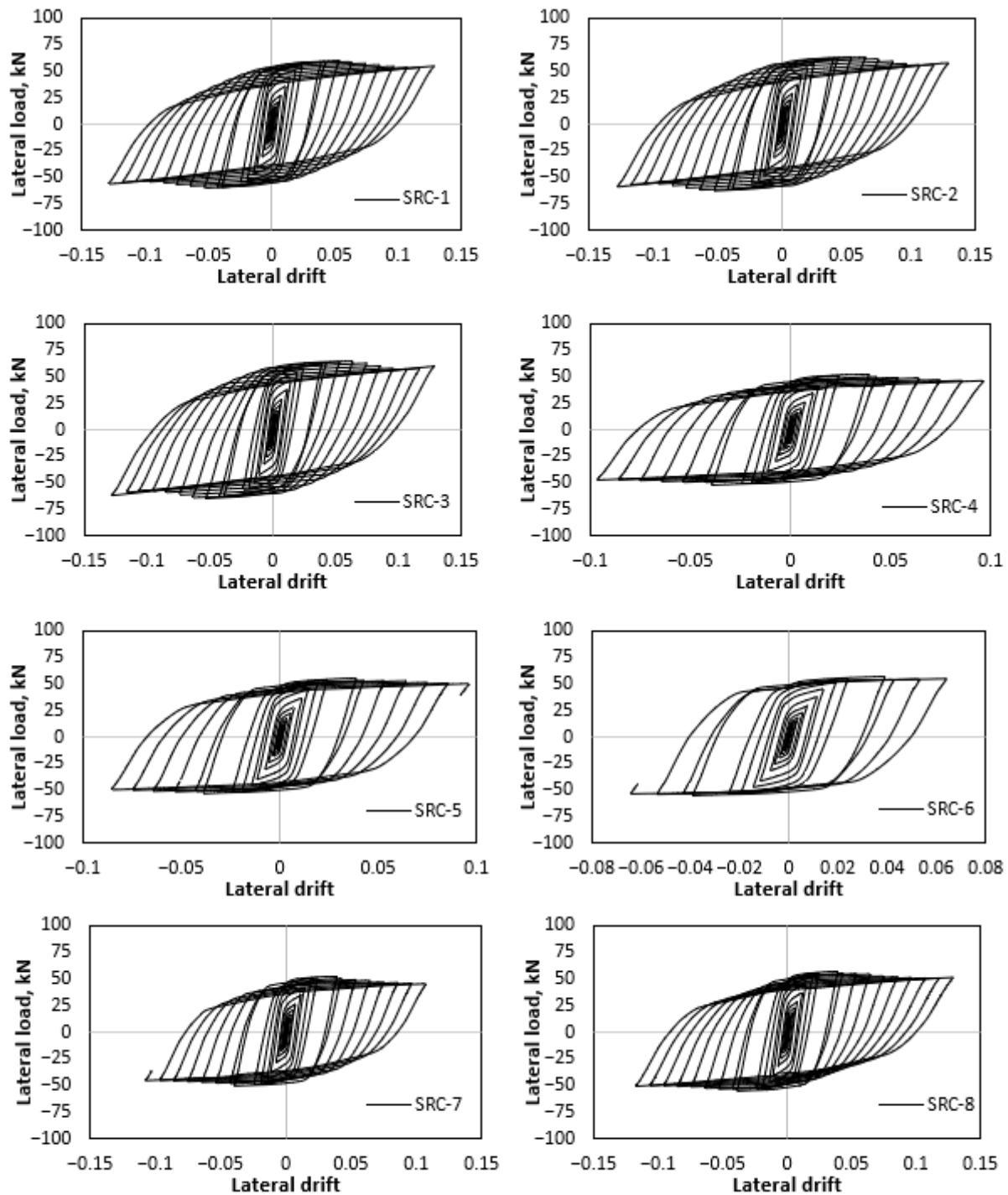


Figure 14. Cont.

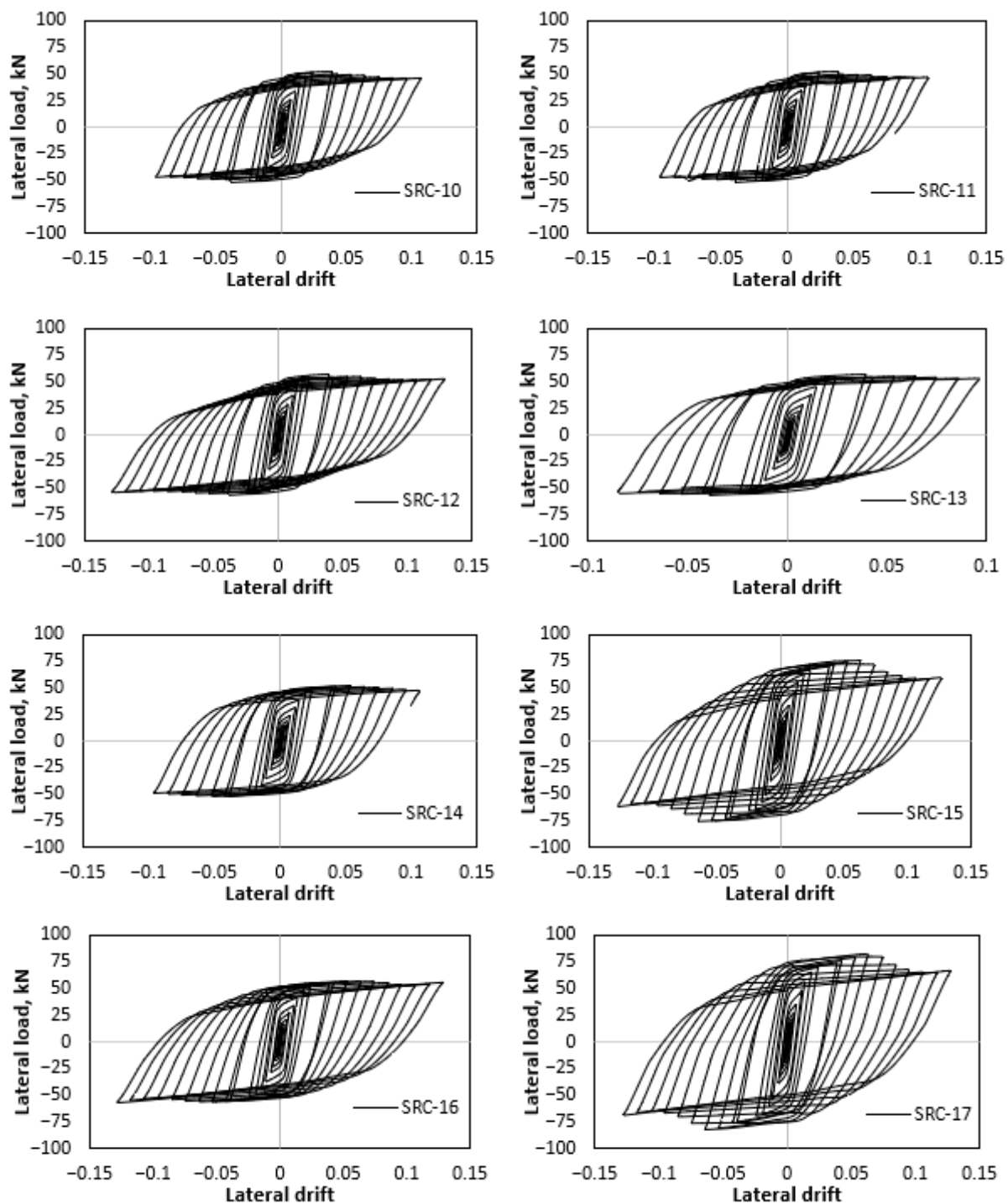


Figure 14. Cyclic lateral load-lateral drift relationships for all models.

An examination of Figure 15 shows that, under the assumption of perfect bond, SRC columns achieve the highest lateral load-carrying capacity at all axial load ratios considered. For a case where the axial load ratio is 0.1, the maximum lateral load was calculated to be 60.01 kN under the assumption of a perfect bond condition, while it was calculated to be 51.85 kN under the friction bond condition model (Table 5). Similar trends can also be observed for cases where the axial load ratio is 0.5 and 0.85.

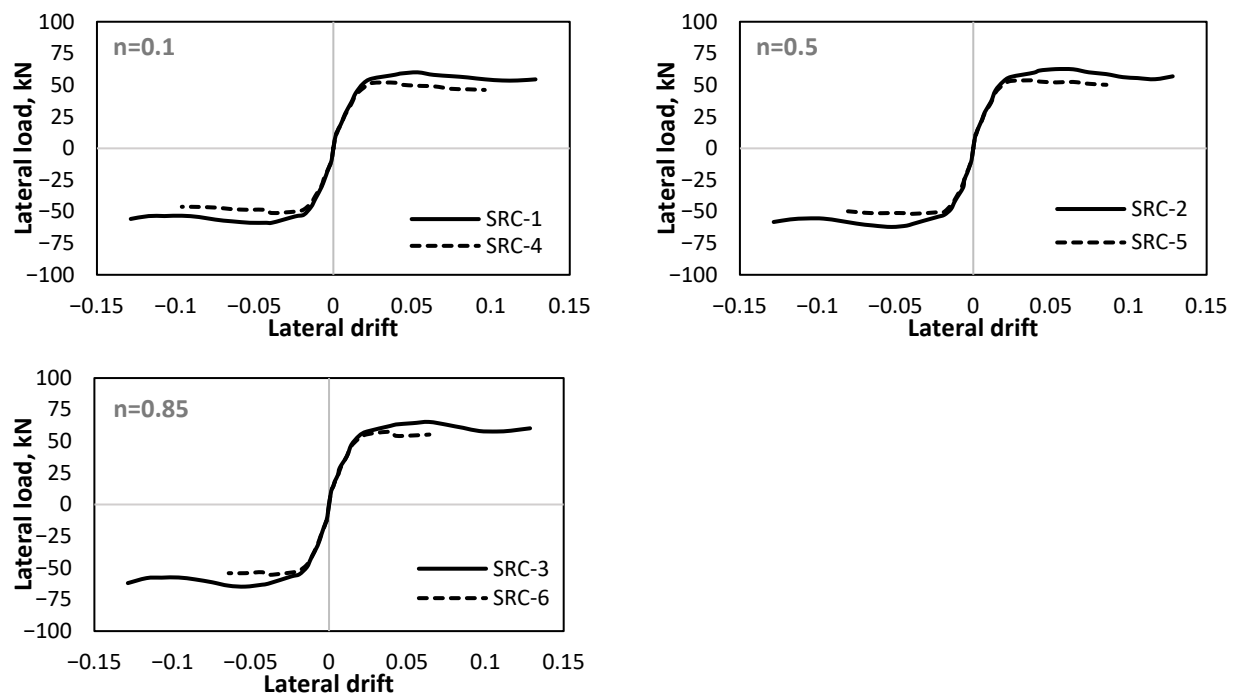


Figure 15. Effect of the type of bond interaction on the lateral load-carrying capacity at different axial load ratios.

Table 5. The relationship between the type of bond interaction and the axial load ratio and seismic performance indicators.

Model Name	Axial Load Ratio (n)	Steel Ratio	Bond Condition	Peak Lateral Load (kN)		Initial Stiffness (kN/mm)		Energy Dissipation Capacity (kJ)
				Push	Pull	Push	Pull	
SRC-1	0.1	2%	Perfect	60.01	−58.95	2.78	−3.07	337.16
SRC-4			Friction	51.85	−51.18	2.75	−3.23	188.06
SRC-2	0.5		Perfect	62.51	−62.22	2.79	−3.09	350.50
SRC-5			Friction	53.68	−51.87	2.75	−3.24	163.37
SRC-3	0.85		Perfect	65.34	−64.91	2.74	−3.19	364.13
SRC-6			Friction	57.51	−55.57	2.87	−3.16	98.12

This decrease in lateral load-carrying capacity can be attributed to the relative reduction at the interface between the structural steel and the concrete. Interface slippage caused by cyclic lateral loads leads to a reduction in composite performance and prevents stresses from being transferred properly across the interface between structural steel and concrete.

In addition to these results, this section of the study also examines the effect of the axial load ratio. Under both the perfect bond and friction bond assumptions, an increase in the axial load ratio generally increased the lateral load-carrying capacity of the SRC column. This result is primarily related to the constraining effect caused by axial compression. In fact, because this confinement effect prevents the crack from propagating and widening, it increases the concrete's compressive strength. Similar observations have also been reported in previous experimental and numerical studies conducted on SRC columns subjected to combined axial and cyclic lateral loading.

Although higher axial load ratios increase the lateral load-carrying capacity of the SRC column, the shape of the hysteresis loops indicates a gradual deterioration in deformation capacity. As a result of this behavior, the increased axial compression intensified local crushing and interface failure, leading to a more pronounced decrease in energy dissipation capacity in models based on frictional bond conditions.

Based on these results, it appears that the assumption of perfect bonding may lead to an overestimation of the seismic performance of SRC columns, and that, if interface conditions are neglected, the lateral load capacity and cyclic stability of SRC columns may be artificially increased at significant axial load levels.

3.1.2. Stiffness Degradation

Figure 16 shows the normalized stiffness reduction curves (K_i/K_0) of SRC columns subjected to cyclic loading, with different types of connection interactions and different axial load ratios. As expected, in all of the SRC-1, SRC-2, SRC-3, SRC-4, SRC-5, and SRC-6 columns, a continuous decrease in stiffness is observed under cyclic lateral load, accompanied by an increasing ratio of lateral displacement due to the effects of interface failure, in addition to cracking and crushing in the concrete and yielding of the reinforcement.

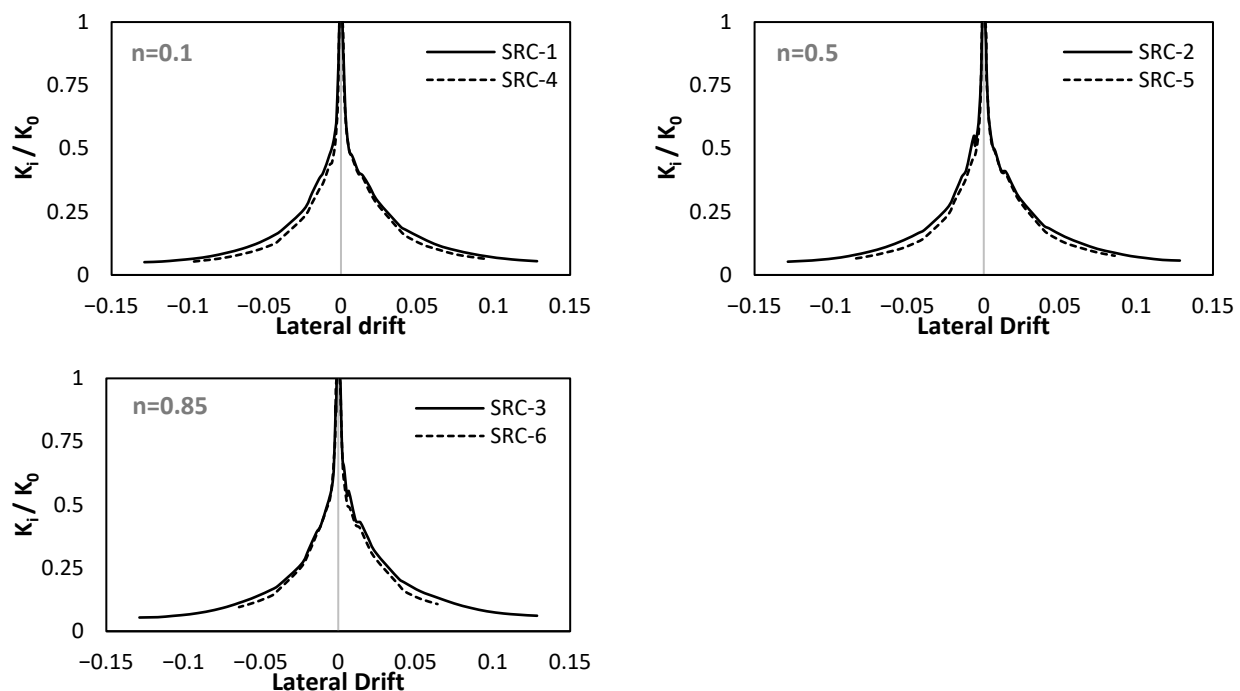


Figure 16. Normalized stiffness degradation of SRC columns subjected to different interface conditions and axial load ratios.

Here, it can be seen that the type of bond interaction at the interface between the structural steel within the SRC column and the surrounding concrete significantly affects the column's stiffness capacity. In fact, for all axial load ratios examined, columns modeled under the assumption of perfect bond interaction exhibited higher normalized stiffness values throughout the loading history compared to those modeled under the assumption of frictional bond interaction (Tables 6–8).

For columns subjected to an axial load ratio of $n = 0.1$, the normalized stiffness values of SRC-1 and SRC-4 are nearly identical at 1% displacement; however, at displacement ratios of 4%, 6%, and 8%, SRC-1 has approximately 14.4%, 15.1%, and 16% higher normalized stiffness values than SRC-4, respectively (Table 6). Accordingly, the results show that while interface slippage has a limited effect in the early stages of cyclic loading, it becomes increasingly pronounced as inelastic deformations increase. Similarly, for columns subjected to an axial load ratio of $n = 0.5$, when the displacement ratio was 8%, the normalized stiffness value of SRC-2 was found to be 26.5% greater than that of SRC-5 (Table 7). At high axial load ratios, however, it is observed that the adverse effects of interface slippage

accelerate the decrease in normalized stiffness due to progressively more severe concrete damage and localized crushing (Table 8).

Table 6. Variation in normalized stiffness values for $n = 0.1$ as a function of the type of bond interaction.

Drift (%)	K _i /K ₀				Reduce %	
	SRC-1		SRC-4			
	Push	Pull	Push	Pull	Push	Pull
1	0.408	0.406	0.401	0.365	1.8	10.1
2	0.348	0.325	0.324	0.281	6.8	13.3
4	0.179	0.175	0.153	0.145	14.4	17.5
6	0.117	0.105	0.099	0.083	15.1	20.9
8	0.084	0.074	0.071	0.069	16.0	6.5
10	0.064	0.058	0.062	0.053	2.7	8.5

Table 7. Variation in normalized stiffness values for $n = 0.5$ as a function of the type of bond interaction.

Drift (%)	K _i /K ₀				Reduce %	
	SRC-2		SRC-5			
	Push	Pull	Push	Pull	Push	Pull
1	0.413	0.415	0.408	0.382	1.2	7.8
2	0.351	0.326	0.338	0.292	3.5	10.5
4	0.196	0.166	0.160	0.132	13.5	20.3
6	0.124	0.110	0.106	0.088	14.8	19.9
8	0.103	0.078	0.076	0.064	26.5	18.0
10	0.067	0.060	-	-	-	-

Table 8. Variation in normalized stiffness values for $n = 0.85$ as a function of the type of bond interaction.

Drift (%)	K _i /K ₀				Reduce %	
	SRC-3		SRC-6			
	Push	Pull	Push	Pull	Push	Pull
1	0.436	0.414	0.420	0.417	3.6	−0.6
2	0.369	0.322	0.341	0.309	7.7	4.2
4	0.207	0.166	0.184	0.141	11.1	14.7
6	0.133	0.112	0.107	0.096	19.4	14.5
8	0.092	0.077	-	-	-	-
10	0.070	0.060	-	-	-	-

The results also show that increasing the axial load ratio tends to increase the sensitivity of stiffness degradation to interface behavior. While a higher axial load contributes to greater locking and increased strength in the early stages of cyclic loading, it also promotes damage accumulation at higher levels of deformation.

These results indicate that the assumption of a perfect bond interaction type may overestimate the stiffness capacity of SRC columns under cyclic loading. It was determined that the differences became more pronounced as the displacement ratios increased and the axial load ratio rose. Accordingly, it is extremely important to incorporate realistic

bonding behavior into the finite element models used to evaluate the seismic performance of SRC columns.

3.1.3. Energy Dissipation Capacity

Energy dissipation capacity is an important indicator of seismic performance in terms of ensuring that the energy generated during an earthquake is dissipated by a structural member through inelastic deformations and distributed to other structural members. Figure 17 and Table 9 show the energy dissipation capacities of SRC columns examined under different types of bond interactions and at different axial load ratios. In this section, the results for the energy dissipation capacity specified for the frictional bond interaction type correspond to a case where the coefficient of friction is 0.4. Since it was determined that the energy dissipation capacity is sensitive to the coefficient of friction, a more detailed parametric analysis of this is presented in Section 3.2.4.

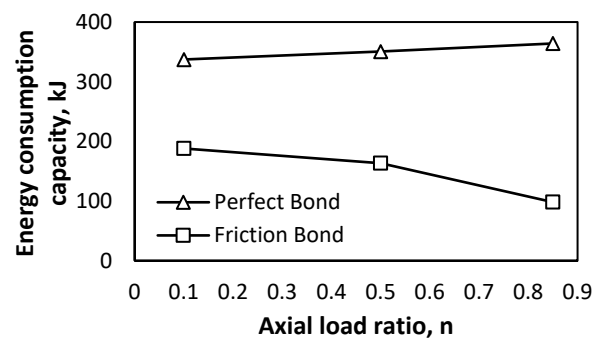


Figure 17. Effect of bond condition and axial load ratio on the cumulative energy dissipation capacity of SRC columns.

Table 9. Comparison of energy consumption capacities for different interface conditions.

Axial Load Ratio	Bond Condition		Reduction (%)
	Perfect	Friction	
0.1	337.16	188.06	44.22
0.5	350.5	163.37	53.39
0.85	364.13	98.12	73.05

In SRC columns modeled using the assumption of a perfect bond-to-member interaction type at all axial load ratios, a higher energy dissipation capacity was obtained compared to models based on frictional bond-to-member interaction types (Figure 15). At axial load ratios of 0.1, 0.5, and 0.85, the energy dissipation capacities of the friction-based bond interaction models were found to be 44.22%, 53.39%, and 73.05% lower, respectively, than those of the corresponding perfect bond interaction models (Table 9).

The energy dissipation capacity of SRC columns decreases as a result of the development of shear effects at the interface between structural steel and concrete. As this slippage at the interface increases, the system's efficiency decreases, and as a result, narrow hysteresis loops reduce the energy dissipation capacity. The effect of bond slippage behavior becomes increasingly significant as damage accumulates and the interface deteriorates. It was also determined that the axial load ratio affects the energy dissipation capacity. In fact, in models of the perfect bond interaction type, as the axial load ratio increased from 0.1 to 0.85, an approximate 8% increase in energy dissipation capacity was calculated. This behavior can be attributed to the enhanced confinement provided by the axial compressive stress, which increases the stability of the cyclic response and delays the onset of severe

damage. However, in models based on frictional bond interactions, the opposite trend emerges. In fact, the energy dissipation capacity decreases by approximately 48% when the axial load ratio increases from 0.1 to 0.85. The combination of high axial pressure and interface slip accelerates local concrete crushing and reduces deformation capacity by promoting the failure of the bond interaction. As a result, the hysteresis loops are gradually narrowing and becoming less stable. Consequently, there is a significant decrease in energy dissipation capacity.

3.2. Effect of Friction Coefficient on Seismic Performance of SRC Columns

In this study, the effect of different friction coefficients (0, 0.2, 0.4, and 0.6) on the seismic performance of SRC columns modeled in accordance with the assumption of a friction bond interaction type was also investigated. The results are presented in Table 10.

Table 10. Performance indicators of SRC columns with different friction coefficients.

Model Name	Axial Load Ratio (n)	Coefficient of Friction	Steel Ratio	Bond Condition	Peak Lateral Load (kN)		Initial Stiffness (kN/mm)		Energy Dissipation Capacity (kJ)	Residual Displacement (mm)	Unloading Stiffness (kN/mm)	Pinching Force Ratio
					Push	Pull	Push	Pull				
SRC-7	0.1	0	2%	Friction	51.98	−50.90	2.75	−3.23	211.57	192.08	0.968	0.841
SRC-10		0.2			51.60	−51.35	2.75	−3.23	207.62	192.37	0.977	0.839
SRC-4		0.4			51.85	−51.18	2.75	−3.23	188.06	192.24	0.981	0.837
SRC-11		0.6			52.40	−51.50	2.75	−3.23	200.20	192.01	0.982	0.828
SRC-8	0.85	0			57.10	−53.89	2.87	−3.16	307.92	187.04	0.982	0.854
SRC-12		0.2			57.53	−56.01	2.87	−3.13	331.47	186.54	0.992	0.851
SRC-6		0.4			57.51	−55.57	2.87	−3.16	98.12	-	-	-
SRC-13		0.6			57.50	−56.06	2.87	−3.15	191.30	185.49	0.990	0.836

Note: The analysis for SRC-6 ($n = 0.85$, $\mu = 0.4$) did not converge over the complete FEMA 461 displacement history applied to the other models; the reported cumulative energy dissipation value therefore reflects a partially completed loading history and should be interpreted with caution when compared with the other models in this table. The additional hysteretic-response indicators were evaluated at the 8% drift cycle. Values are not reported for SRC-6 because this model did not complete the corresponding loading cycle.

3.2.1. Load-Carrying Capacity and Initial Stiffness

The results in Table 10 indicate that the friction coefficient has a limited influence specifically on the peak lateral load-carrying capacity and initial stiffness of the investigated SRC columns. This observation should not be interpreted as indicating an equally limited influence on the overall hysteretic response, which is further evaluated in terms of residual displacement, unloading stiffness, and pinching behavior. In fact, in models where the axial load ratio is 0.1, the lateral load capacity ranges from 51.60 kN to 52.40 kN in compression and from 50.90 kN to 51.50 kN in tension, while the initial stiffness remains constant at 2.75 kN/mm in compression and 3.23 kN/mm in tension. Although a similar trend is observed in models with an axial load ratio of 0.85, the lateral load capacity and initial stiffness in these models showed only very slight changes despite variations in the coefficient of friction.

3.2.2. Residual Displacement, Unloading Stiffness, and Pinching Behavior

To further quantify the influence of the friction coefficient on the hysteretic-loop characteristics, residual displacement, unloading stiffness, and pinching force ratio were evaluated at the 8% drift cycle, as summarized in Table 10. At ($n = 0.10$), the residual displacement remained nearly unchanged, ranging from 192.01 to 192.37 mm as the friction coefficient varied from 0 to 0.6. Over the same range, the unloading stiffness increased slightly from 0.968 to 0.982 kN/mm, while the pinching force ratio decreased from 0.841 to 0.828.

A similar response was observed at ($n = 0.85$). Excluding SRC-6, which did not complete the 8% drift cycle, the residual displacement decreased from 187.04 mm to 185.49 mm, whereas the unloading stiffness remained within a relatively narrow range of 0.982–0.992 kN/mm. The pinching force ratio decreased from 0.854 to 0.836. These results confirm that variations in interface friction modify the hysteretic-loop characteristics, particularly the unloading response and pinching behavior, although the quantified changes in these indicators remain moderate within the investigated friction range.

3.2.3. Stiffness Degradation

As shown in Figure 18, the normalized stiffness of all SRC columns progressively decreased with increasing drift, indicating the accumulation of damage under cyclic loading. For both axial load ratios, the curves corresponding to different friction coefficients remained relatively close throughout the loading history, suggesting that the interface friction coefficient had a limited influence on the overall stiffness degradation within the investigated range. Although local differences among the curves became more apparent at larger drift levels, no consistent monotonic relationship was observed between the friction coefficient and the normalized stiffness ratio. This behavior indicates that, once relative slip is permitted at the steel–concrete interface, stiffness degradation is governed predominantly by the progressive inelastic response of the composite member, while variations in frictional resistance primarily produce secondary modifications to the stiffness response.

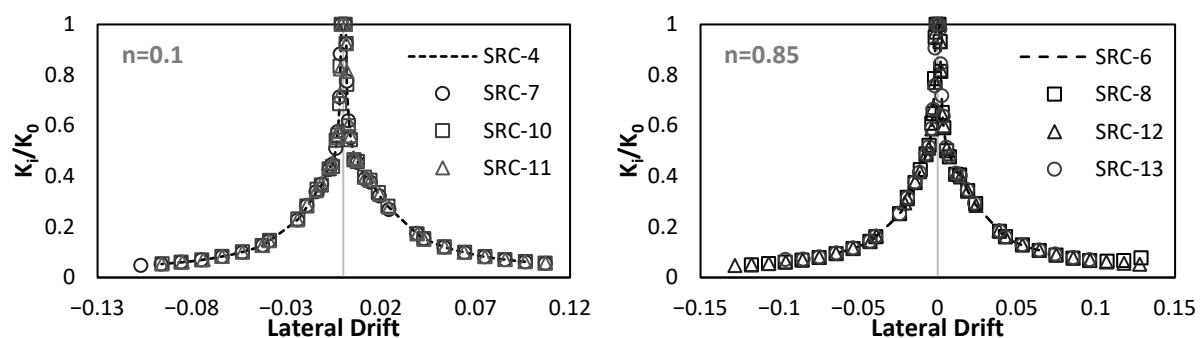


Figure 18. Normalized stiffness degradation curves of SRC columns with different coefficients of friction.

3.2.4. Energy Dissipation Capacity

As shown in Figure 19, the influence of the friction coefficient on energy dissipation capacity was highly dependent on the axial load ratio. For specimens subjected to $n = 0.1$, only moderate variation was observed. For $n = 0.85$, the SRC-6 model ($\mu = 0.4$) exhibited a markedly lower energy dissipation value than the neighboring friction levels; however, this analysis did not converge over the full prescribed displacement history, so the apparent reduction reflects an incomplete loading record for this specific case rather than a genuine mechanical trend. Consequently, this value is excluded from trend interpretation, and the comparison across friction coefficients at $n = 0.85$ should rely primarily on $\mu = 0, 0.2$, and 0.6 , which completed the full loading protocol.

The nonlinear variation in energy dissipation with the friction coefficient can be attributed to the changing balance between interfacial slip and shear transfer at the steel–concrete interface. Increasing the friction coefficient enhances the frictional shear resistance and tends to restrain relative slip between the embedded structural steel and surrounding concrete; however, this does not necessarily produce a proportional increase in the total energy dissipated by the SRC column. Instead, changes in interface restraint modify the distribution of deformation and damage among interfacial slip, concrete cracking and crushing, and steel plasticity, resulting in a nonlinear variation in cumulative energy

dissipation. This effect becomes more pronounced at the higher axial load ratio because the increased compressive force produces greater normal contact pressure at the steel–concrete interface, thereby amplifying the frictional resistance associated with a given friction coefficient. Consequently, variations in the friction coefficient have a stronger influence on the interface response and the resulting energy-dissipation mechanism at ($n = 0.85$) than at ($n = 0.10$).

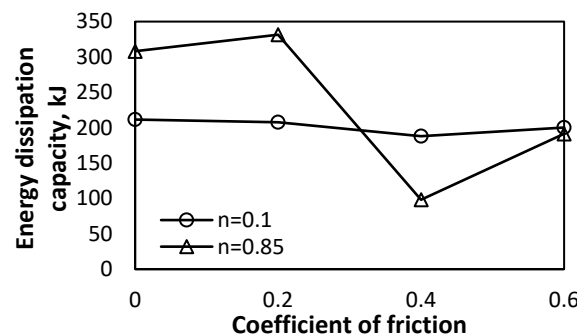


Figure 19. Effect of friction coefficient on the cumulative energy dissipation capacity of SRC columns subjected to cyclic loading.

It is noteworthy that a local minimum in energy dissipation capacity at $\mu = 0.4$ is observed for both axial load ratios (Table 10): at $n = 0.1$, SRC-4 ($\mu = 0.4$) shows a mild reduction relative to the neighboring friction levels (≈ 9 – 11% lower), whereas at $n = 0.85$, SRC-6 ($\mu = 0.4$) exhibits a much more severe reduction linked to the convergence difficulty discussed above. This recurring pattern may be associated with $\mu = 0.4$ corresponding to a transitional stick–slip regime at the steel–concrete interface, in which the contact status alternates frequently between sticking and sliding during cyclic loading, increasing the numerical demand on the contact algorithm. Under the low axial load ratio ($n = 0.1$), the resulting contact instability manifests only as a modest reduction in dissipated energy, since the normal contact pressure—and hence the frictional resistance developed at the interface—remains limited. Under the high axial load ratio ($n = 0.85$), however, the substantially higher normal contact pressure amplifies this instability, which is considered the primary reason why the SRC-6 analysis, unlike the other models in the parametric matrix, did not converge over the complete FEMA 461 [38] displacement history.

3.3. Effect of Steel Ratio on Seismic Performance of SRC Columns

The effect of the longitudinal reinforcement ratio on the cyclic lateral load behavior of SRC columns was evaluated by comparing columns with reinforcement ratios of 1.5%, 2.0%, and 3.0% under axial load ratios of 0.1 and 0.85 (Figure 20). The results obtained show that increasing the reinforcement ratio significantly improves the seismic performance of SRC columns.

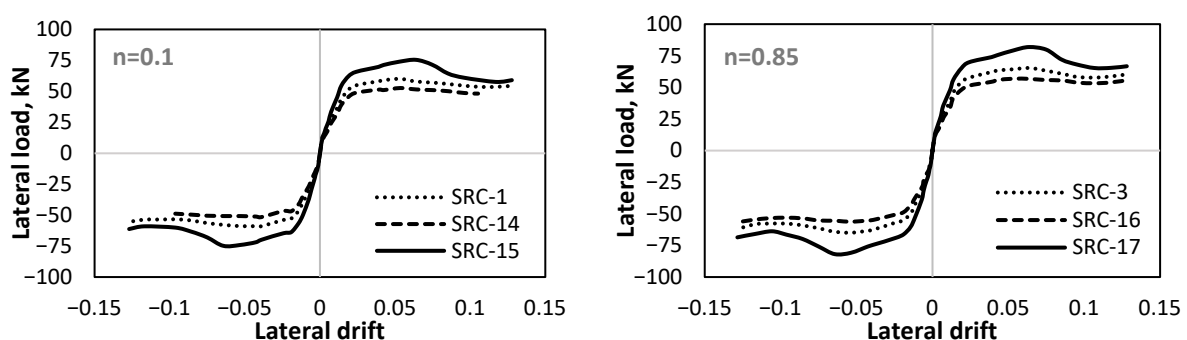


Figure 20. Effect of low, medium, and high reinforcement ratios on lateral load-carrying capacity.

3.3.1. Lateral Load and Initial Stiffness Response

For columns subjected to an axial load ratio of 0.1, increasing the reinforcement ratio from 1.5% to 3.0% increased the lateral load-bearing capacity by approximately 45%. Similarly, an increase of approximately 46% was observed for columns subjected to an axial load ratio of 0.85 (Table 11). These results indicate that the reinforcement ratio is one of the primary parameters determining the seismic performance of SRC columns under cyclic lateral loading.

Table 11. Changes in seismic performance indicators as a function of reinforcement ratio.

Model Name	Axial Load Ratio (n)	Steel Ratio	Bond Condition	Peak Lateral Load (kN)		Initial Stiffness (kN/mm)		Energy Dissipation Capacity (kJ)
				Push	Pull	Push	Pull	
SRC-14	0.1	1.5%	Perfect	52.68	−51.64	2.74	−2.98	199.12
SRC-1		2%		60.01	58.95	2.78	−3.07	337.16
SRC-15		3%		75.42	−74.82	2.87	−3.26	391.36
SRC-16	0.85	1.5%		56.86	−56.28	2.69	−3.10	331.47
SRC-3		2%		65.34	−64.91	2.74	−3.19	364.13
SRC-17		3%		81.81	−81.89	2.83	−3.44	427.83

In contrast, the effect of the reinforcement ratio on the initial stiffness was relatively limited. Increasing the reinforcement ratio from 1.5% to 3.0% resulted in an approximate 5% increase in stiffness for both axial load levels (Table 11). This result shows that the elastic response is initially governed by the overall stiffness of the composite cross-section, while the reinforcement ratio becomes increasingly important after the onset of inelastic deformation.

3.3.2. Energy Dissipation Capacity

Increasing the reinforcement ratio has also significantly improved the energy dissipation capacity. In columns subjected to an axial load ratio of 0.1, the energy dissipation capacity increased from 199.12 kJ to 391.36 kJ, which corresponds to an increase of approximately 97%. In columns subjected to an axial load ratio of 0.85, the dissipated energy increased from 331.47 kJ to 427.83 kJ, representing an increase of approximately 29%. As shown in Figure 21, increasing the reinforcement ratio significantly enhances the energy dissipation capacity, particularly at low axial load levels. The improvement in energy dissipation capacity can be attributed to the increased plastic deformation capacity provided by the longitudinal reinforcement, which promotes more stable hysteresis behavior during cyclic loading.

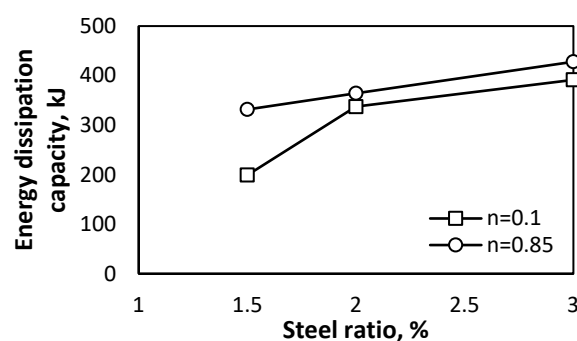


Figure 21. Effect of reinforcement ratio on cumulative energy dissipation capacity of SRC columns.

3.3.3. Stiffness Degradation

Normalized stiffness reduction curves further demonstrate the beneficial effect of an increased reinforcement ratio (Figure 22).

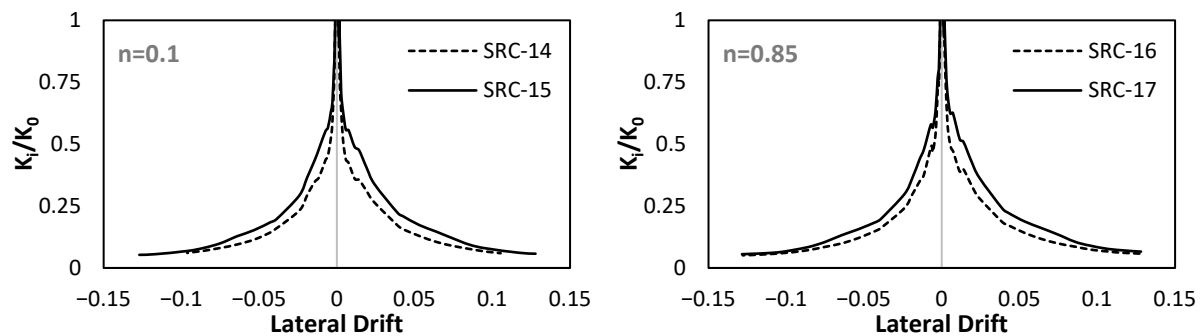


Figure 22. Normalized stiffness degradation curves of SRC columns with different reinforcement ratios.

For all lateral displacement ratios, columns with a 3.0% reinforcement ratio have higher normalized stiffness values than corresponding columns with a 1.5% reinforcement ratio. The difference becomes more pronounced at higher lateral displacement rates, indicating that an increased reinforcement ratio delays damage accumulation and improves the maintenance of stiffness during cyclic loading.

4. Conclusions and Engineering Implications

This study investigated the influence of steel–concrete interface behavior on the cyclic response of steel-reinforced concrete (SRC) columns using three-dimensional nonlinear finite element analysis. The numerical model was established in ABAQUS using appropriate solid and truss elements, nonlinear constitutive models for concrete and steel, and two alternative steel–concrete interface formulations: perfect bonding and frictional contact. The model was verified against available experimental results and reproduced the overall lateral response with good agreement, including a difference of approximately 0.22% in peak lateral load. The established modeling framework was subsequently used for the parametric investigation.

The friction coefficients of $\mu = 0.0, 0.20, 0.40$, and 0.60 were adopted to represent different levels of frictional resistance that may develop at the steel–concrete interface following loss of adhesive bonding. These values were varied systematically to examine the sensitivity of the cyclic response to interface friction while maintaining the other principal modeling parameters unchanged. The selected values should therefore be regarded as sensitivity-analysis levels rather than experimentally calibrated interface parameters.

The principal findings of the three parametric groups defined in Table 2 can be summarized as follows.

The results demonstrate that the steel–concrete interface condition has a substantial influence on the cyclic response of SRC columns. Compared with the frictional-interface models, the perfect-bond models generally provided higher lateral resistance, better stiffness retention, and greater cumulative energy dissipation. The influence of interface behavior became more pronounced with increasing axial load ratio, particularly in terms of cumulative energy dissipation. These findings indicate that the assumption of perfect bonding can significantly affect the predicted cyclic response when interface slip is not explicitly considered.

The interface friction coefficient had a comparatively limited influence on peak lateral resistance and initial stiffness within the investigated range, whereas its influence on cumulative energy dissipation was more noticeable. This indicates that the role of interface friction is more closely associated with the post-bonding cyclic response than with the initial load-carrying characteristics of the SRC columns. The analysis with $\mu = 0.40$ at an axial load ratio of 0.85 did not complete the prescribed loading history; therefore, this case was not used to establish a general trend for the effect of friction coefficient.

Increasing the longitudinal reinforcement ratio enhanced the lateral load-carrying capacity and cumulative energy dissipation of the SRC columns, while its influence on initial stiffness was comparatively limited within the investigated range. The strengthening effect was observed at both low and high axial load ratios, although the magnitude of the increase in energy dissipation varied with the axial load level.

The findings of this study demonstrate that steel–concrete interface behavior should not be regarded as a secondary modeling detail in nonlinear finite element simulations of SRC columns. Although the perfect bond assumption offers computational simplicity, it may produce unconservative predictions of seismic performance by overestimating composite action throughout cyclic loading. For practical nonlinear finite element modeling, the present results suggest that the perfect bond assumption should not be used as the sole interface representation when cyclic energy dissipation is a governing response parameter, particularly at moderate-to-high axial load ratios. Within the investigated parameter range, the perfect bond model overestimated cumulative energy dissipation relative to the frictional interface model by approximately 44%, 53%, and 73% at axial load ratios of 0.10, 0.50, and 0.85, respectively. Accordingly, analyses based solely on perfect bond should be interpreted with increasing caution as the axial load ratio increases. Where experimentally calibrated bond–slip data are unavailable, a frictional contact formulation allowing interface slip and separation is recommended as a sensitivity model alongside the conventional perfect bond assumption. The differences between the two interface idealizations can then be used to quantify the uncertainty associated with the assumed degree of composite action.

These percentage differences should not be interpreted as universal correction factors, since they were obtained for the geometry, material properties, loading protocol, and interface formulations investigated in the present study. Rather, they provide quantitative bounds on the sensitivity observed in the analyzed SRC columns and demonstrate the magnitude of error that may arise when interface slip is completely suppressed.

5. Limitations and Future Research

The present investigation was intentionally limited to perfect bond and Coulomb friction interface formulations in order to isolate the influence of interface behavior using a numerically robust modeling framework. More sophisticated cohesive interface constitutive models, capable of explicitly representing bond degradation and progressive debonding, were not considered because experimentally calibrated bond–slip parameters were unavailable. In addition, the study focused on a single column geometry, one embedded steel section, and quasi-static cyclic loading conditions.

A further limitation concerns the interaction between the longitudinal reinforcement and the surrounding concrete. In the present study, a perfect-bond assumption was adopted for the reinforcement–concrete interface, while the effects of bond-slip, local debonding, and cyclic degradation of the reinforcement–concrete interaction were not explicitly modeled. Such an assumption may lead to a comparatively fuller and more stable hysteretic response than would be obtained when reinforcement bond-slip and interface degradation are explicitly considered, particularly under cyclic loading. Therefore, the hysteretic response and associated energy dissipation predicted by the present finite element models should be interpreted with this modeling assumption in mind.

Future research should incorporate experimentally calibrated cohesive interface models, investigate different steel section configurations and concrete strength classes, and validate the proposed modeling strategy against additional cyclic experimental data. Extending the analyses to dynamic earthquake loading and structural system-level behavior would further improve understanding of interface mechanisms and contribute to the devel-

opment of more reliable design recommendations for steel-reinforced concrete structures subjected to seismic actions.

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References

1. Uy, B. Strength of short concrete filled high strength steel box columns. *J. Constr. Steel Res.* **2001**, *57*, 113–134. [\[CrossRef\]](#)
2. Lai, B.L.; Bao, R.L.; Zhang, M.Y.; Wang, Y.H.; Liew, J.R. Evaluation on the static and seismic performance of steel reinforced concrete composite columns with high strength materials. *J. Build. Eng.* **2023**, *79*, 107886. [\[CrossRef\]](#)
3. Zhao, X.; Wen, F.; Yan, S. Behavior of multi-steel reinforced concrete columns with encased steel H-sections. *Eng. Struct.* **2022**, *264*, 114463. [\[CrossRef\]](#)
4. Liu, X.; Zeng, L.; Xiao, Y.; Liu, Y.; Zhang, H.; Cao, Y. Experimental and Numerical Investigation on the Cyclic Behavior of SRC Columns considering Unsymmetrical Confinement Effect. *KSCE J. Civ. Eng.* **2022**, *26*, 1880–1893. [\[CrossRef\]](#)
5. Gautham, A.; Sahoo, D.R. Behavior of steel-reinforced composite concrete columns under combined axial and lateral cyclic loading. *J. Build. Eng.* **2021**, *39*, 102305. [\[CrossRef\]](#)
6. Jiang, L.; Bai, G. Experimental study on cumulative damage behavior of steel-reinforced concrete columns. *Adv. Civ. Eng.* **2020**, *2020*, 5281725. [\[CrossRef\]](#)
7. Yan, S.; Wen, F.; Zhao, X.; Chen, Y. Seismic behaviour of SRC columns with high encased steel ratio. *Steel Compos. Struct.* **2025**, *54*, 157.
8. Munoz, P.R.; Hsu, C.T.T. Behavior of biaxially loaded concrete-encased composite columns. *J. Struct. Eng.* **1997**, *123*, 1163–1171. [\[CrossRef\]](#)
9. Chen, C.C.; Chen, C.C.; Hoang, T.T. Role of concrete confinement of wide-flange structural steel shape in steel reinforced concrete columns under cyclic loading. *Eng. Struct.* **2016**, *110*, 79–87. [\[CrossRef\]](#)
10. Xiang, S.; Zeng, L.; Liu, Y.; Mo, J.; Ma, L.; Zhang, J.; Chen, J. Experimental study on the dynamic behavior of T-shaped steel reinforced concrete columns under impact loading. *Eng. Struct.* **2020**, *208*, 110307. [\[CrossRef\]](#)
11. Harajli, M.H. Bond behavior in steel fiber-reinforced concrete zones under static and cyclic loading: Experimental evaluations and analytical modeling. *J. Mater. Civ. Eng.* **2010**, *22*, 674–686. [\[CrossRef\]](#)
12. Chiu, L.N.; Falzon, B.G.; Boman, R.; Chen, B.; Yan, W. Finite element modelling of composite structures under crushing load. *Compos. Struct.* **2015**, *131*, 215–228. [\[CrossRef\]](#)
13. Yang, Y.; Wang, Y.; Fu, F.; Liu, J. Static behavior of T-shaped concrete-filled steel tubular columns subjected to concentric and eccentric compressive loads. *Thin-Walled Struct.* **2015**, *95*, 374–388. [\[CrossRef\]](#)
14. Xiang, S.; Zeng, L.; Zhang, J.C.; Chen, J.; Liu, Y.H.; Cheng, G.Y.; Mo, J.X. A DIC-based study on compressive responses of concrete after exposure to elevated temperatures. *Materials* **2019**, *12*, 2044. [\[CrossRef\]](#) [\[PubMed\]](#)
15. Shanmugam, N.E.; Lakshmi, B. State of the art report on steel–concrete composite columns. *J. Constr. Steel Res.* **2001**, *57*, 1041–1080. [\[CrossRef\]](#)
16. Ellobody, E.; Young, B. Nonlinear analysis of concrete-filled steel SHS and RHS columns. *Thin-Walled Struct.* **2006**, *44*, 919–930. [\[CrossRef\]](#)
17. Lai, B.; Liew, J.Y.R.; Xiong, M. Experimental study on high strength concrete encased steel composite short columns. *Constr. Build. Mater.* **2019**, *228*, 116640. [\[CrossRef\]](#)
18. Shen, Z.; Liu, B.; Zhou, G. Stressing state analysis of SRC column with modeling test and finite element model data. *Appl. Sci.* **2022**, *12*, 8866. [\[CrossRef\]](#)
19. Wu, C.; Song, H.; Mou, B.; Qu, Z.; Zhu, Z.; Yu, H. Development and evaluation of a novel prefabricated SRC column-steel beam composite joint with artificial plastic hinges. In *Structures*; Elsevier: Amsterdam, The Netherlands, 2025; Volume 79, p. 109655. [\[CrossRef\]](#)
20. Wang, D.; Liu, Q.; Song, Y.; Zhu, Y.; Zhang, Y. Experimental and numerical study on seismic behaviors of SRC column-RC slab joints in NPP. *Nucl. Eng. Technol.* **2025**, *57*, 103176. [\[CrossRef\]](#)
21. Naghipour, M.; Ahmadi, M.; Nematzadeh, M. Effect of concrete confinement level on load-bearing capacity of steel-reinforced concrete (SRC) columns under eccentric loading: Experiment and FEA model. In *Structures*; Elsevier: Amsterdam, The Netherlands, 2022; Volume 35, pp. 202–213. [\[CrossRef\]](#)
22. Luu, X.B.; Kim, S.K. Finite element modeling of interface behavior between normal concrete and ultra-high-performance fiber-reinforced concrete. *Buildings* **2023**, *13*, 950. [\[CrossRef\]](#)

23. Wang, X.; Liu, Y.; Yang, F.; Lu, Y.; Li, X. Effect of concrete cover on the bond-slip behavior between steel section and concrete in SRC structures. *Constr. Build. Mater.* **2019**, *229*, 116855. [\[CrossRef\]](#)
24. Gautham, A.; Sahoo, D.R. Performance of SRC column-RC beam joints under combined axial and cyclic lateral loadings. *Eng. Struct.* **2022**, *260*, 114218. [\[CrossRef\]](#)
25. Ren, R.; Qi, L.; Xue, J.; Zhang, X. Cyclic bond property of steel reinforced recycled concrete (SRRC) composite structure. *Constr. Build. Mater.* **2020**, *245*, 118435. [\[CrossRef\]](#)
26. Dassault Systèmes Simulia Corp. *Abaqus/CAE*, version 6.12; Dassault Systèmes Simulia Corp.: Johnston, RI, USA, 2017.
27. TS 500; Requirements for Design and Construction of Reinforced Concrete Structures. Turkish Standards Institution: Ankara, Türkiye, 2000.
28. Ersoy, U.; Özcebe, G. *Betonarme*; Evrim Yayınevi: Ankara, Türkiye, 2018.
29. Gan, D.; Guo, L.; Liu, J.; Zhou, X. Seismic behavior and moment strength of tubed steel reinforced-concrete (SRC) beam-columns. *J. Constr. Steel Res.* **2011**, *67*, 1516–1524. [\[CrossRef\]](#)
30. EN 1992-1-1:2004; Eurocode 2: Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings. European Committee for Standardization (CEN): Brussels, Belgium, 2004.
31. Comité Euro-International Du Béton. *Fire Design of Concrete Structures-in Accordance with CEB/FIB Model Code 90*; CEB Bulletin D'Information, No. 208; Comité Euro-International Du Béton: Lausanne, Switzerland, 1991.
32. Genikomsou, A.S.; Polak, M.A. Finite element analysis of punching shear of concrete slabs using damaged plasticity model in ABAQUS. *Eng. Struct.* **2015**, *98*, 38–48. [\[CrossRef\]](#)
33. Çolakoğlu, H.E.; Hüsem, M. Investigation of the cyclic load behavior of reinforced concrete frames exposed to high temperatures using the finite element method. *Sigma* **2025**, *43*, 857–877. [\[CrossRef\]](#)
34. Parthasarathi, N.; Satyanarayanan, K.S. Progressive collapse behavior of reinforced concrete frame exposed to high temperature. *J. Struct. Fire Eng.* **2020**, *12*, 110–124. [\[CrossRef\]](#)
35. Çolakoğlu, H.E.; Hüsem, M. Seismic behavior of a RC column subjected to cyclic lateral loading after high temperature. *J. Innov. Eng. Nat. Sci.* **2026**, *6*, 275–294. [\[CrossRef\]](#)
36. Öztemel, M.; Sümer, Y. Numerical Investigation of the Structural Behavior of Reinforced Concrete Deep Beams Strengthened with Fiber Reinforced Polymer (FRP) Reinforcing Bars Using the Finite Element Method. *Buildings* **2026**, *16*, 1521. [\[CrossRef\]](#)
37. Sümer, Y.; Öztemel, M. Investigation of the Effect of GFRP Reinforcement Bars on the Flexural Strength of Reinforced Concrete Beams Using the Finite Element Method. *Fibers* **2025**, *13*, 125. [\[CrossRef\]](#)
38. FEMA 461; Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components. Federal Emergency Management Agency: Washington, DC, USA, 2007.
39. Zhang, L.; Wang, W.; Cai, J.; Lou, T.; Hou, H.; Lou, F. Seismic damage assessment for partially encased composite columns based on drift capacity. In *Structures*; Elsevier: Amsterdam, The Netherlands, 2025; Volume 81, p. 110354. [\[CrossRef\]](#)
40. Ji, S.H.; Wang, W.D.; Xian, W.; Shi, Y.L. Cyclic and monotonic behaviour of steel-reinforced concrete-filled steel tubular columns. *Thin-Walled Struct.* **2023**, *185*, 110644. [\[CrossRef\]](#)
41. Jiang, Y.D.; Silva, A.; Castro, J.M.; Chan, T.M.; Monteiro, R. Experimental study and numerical assessment of the flexural behaviour of square and rectangular CFST members under monotonic and cyclic loading. *Key Eng. Mater.* **2018**, *763*, 804–811. [\[CrossRef\]](#)

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