



ARTICLE

Numerical Investigation of the Seismic Performance of RCS Connections with a Novel Simplified Detailing

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ABSTRACT: RC column-steel beam (RCS) structural systems have been extensively studied over the past few decades because they combine the advantages of steel beams and RC columns. The seismic performance of the system is strongly governed by joint detailing. Proper joint detailing is essential for achieving both satisfactory mechanical performance and constructability. A novel joint configuration, referred to as the whole column-section diaphragm joint, has been proposed and experimentally validated in previous studies, showing favorable seismic performance together with a simplified fabrication process. The objective of this study is to develop a detailed three-dimensional finite element model to investigate the seismic behavior of this proposed RCS joint configuration. The modeling method was validated against six cyclic tests of interior RCS joints with whole column-section diaphragms. The model reproduced the global hysteretic response, peak resistance, stiffness characteristics, and local joint shear failure observed in the experiments. Parametric analyses were conducted to examine the effects of axial compression ratio and face bearing plate (FBP) thickness and to derive design-oriented recommendations. The results indicate that, within the investigated parameter range, an axial compression ratio below 0.3 is beneficial for maintaining stable joint behavior, whereas increasing the parallel FBP thickness provides limited strength enhancement. Existing design provisions from ASCE, AIJ, and CECS 347:2013 were further evaluated. Based on experimental and numerical results, a modified CECS-based expression was proposed by modifying the concrete contribution to incorporate the effect of axial compression ratio, improving the prediction accuracy of joint shear strength for the investigated joint type.

KEYWORDS: Reinforced concrete column-steel beam; beam-column joints; axial compression ratio; face bearing plate; seismic performance

1 Introduction

RC column-steel beam (RCS) hybrid frames have attracted increasing attention because they make efficient use of both steel and concrete and are well suited to prefabricated construction [1,2]. Within such systems, beam-column joints form the critical load-transfer region. The two commonly used joint configurations, namely through-beam and through-column joints, are illustrated in Fig. 1. Early studies identified two principal failure modes: panel shear failure and bearing failure. Panel shear failure is associated with steel web yielding and severe concrete spalling within the joint core, whereas bearing failure is characterized by rigid-body rotation of the steel beam caused by concrete crushing above and below the beam flanges [3,4].

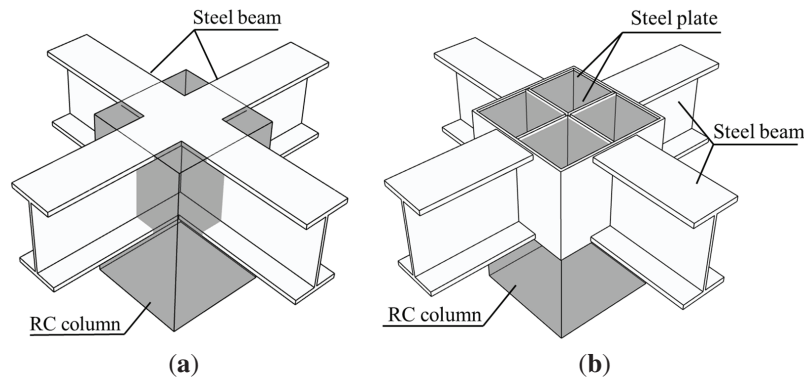


Figure 1: Two typical types of RCS joints. (a) Through beam joint; (b) Through column joint.

To mitigate these undesirable failure modes and improve seismic performance, various strengthening details have been proposed, including face bearing plates, cover plates, headed studs, and steel fiber reinforced concrete. Previous studies have shown that these measures can effectively enhance the strength, ductility, and energy dissipation capacity of RCS joints [5–8]. Herdiansah et al. [9] developed a void-web RCS joint using additional bearing plates, extended band plates, and U-shaped stirrups, showing that these details improved joint panel confinement and reduced damage under cyclic loading. Khaloo and Bakhtiari Doost [10] investigated precast RCS connections with extended FBPs and cover plates, showing that these details improved joint shear resistance and seismic performance.

More recently, the research focus has shifted toward balancing mechanical performance with constructability. For instance, Alizadeh et al. [11] applied self-consolidating concrete to connection casting, significantly improving construction efficiency and ensuring the integrity of the concrete-steel interface. Lee et al. [12] developed four simplified interior joints using FBPs, transverse beams, and headed studs, demonstrating that simplified detailing can still provide sufficient deformation capacity for moderate seismic demands. In addition, Mirghaderi et al. [13] proposed a novel joint using parallel steel beams passing through the column faces to maintain structural continuity and reported stable hysteretic behavior and high energy dissipation up to a 7% story drift. Joju et al. [14] developed through-column RCS joints for circular high-strength RC columns, showing improved constructability and reduced reinforcement congestion compared with traditional through-beam joints. Demountable joints with bolted end plates have also attracted considerable attention due to their ease of construction [15]. Chen et al. [16] proposed a high-strength bolt end plate RCS connection for prefabricated hybrid frames, and the tested specimens exhibited good structural integrity and seismic behavior. Li et al. [17] further studied demountable diagonal-connection RCS joints, showing stable hysteretic behavior and improved replaceability.

To address fabrication difficulties while avoiding brittle bearing failure, Tao et al. [18] proposed a joint detail with a whole column-section diaphragm. In this configuration, the beam web is interrupted at the beam-column interface, which facilitates concrete casting and stirrup arrangement. At the same time, the enlarged bearing area adjacent to the beam flanges reduces local bearing stress and helps suppress premature bearing failure.

Considerable effort has also been devoted to the numerical simulation of RCS joints. Cheng and Chen [1] used the nonlinear program DRAIN-2DX to simulate force-displacement responses, accounting for composite interaction between beams and slabs and shear distortion in the joint panel zone. Shen [19] employed ANSYS to analyze the static behavior and stress redistribution of exterior RCS joints. Zhang [20] developed ABAQUS models to investigate stress evolution and load-displacement response for different joint

configurations, clarifying the influence of specific detailing on shear resistance. Based on parametric FE analyses, Li et al. [21] proposed simplified lateral load-displacement hysteretic models for frame-level seismic analysis of RCS structures. Jafari et al. [22] developed a numerical model for RCS joints with tube plates, providing insight into the contribution of tube plates to joint performance. Li et al. [23] developed a validated nonlinear FE model for simplified RCS joints and showed that a modified Kanno-based method predicted the bearing capacity within 5% error.

Previous numerical studies have demonstrated that finite element analysis can accurately capture the response of RCS joints. However, most of them focus on monotonic behavior. Existing cyclic studies are still limited, and the reported response is mostly governed by steel beam yielding rather than brittle joint shear failure. Consequently, the numerical simulation of RCS joints with joint shear failure mode remains insufficient. Moreover, the whole column-section diaphragm detail considered in this study differs fundamentally from conventional through-beam and through-column RCS joints. The internal load-transfer mechanism affected by a truncated steel web and a whole column-section diaphragm is different from that of traditional RCS joint configurations. Therefore, both a dedicated FE investigation and a reassessment of the applicability of existing design formulas are needed for this type of joint.

Accordingly, this study has two main objectives. The first is to establish a reliable three-dimensional finite element model for RCS joints with whole column-section diaphragms. The second is to use this model to evaluate the effects of axial compression ratio and parallel FBP thickness on seismic performance. On this basis, the applicability of existing design provisions is assessed, and a modified CECS-based shear-strength expression is proposed.

2 Experimental Program

The whole column-section diaphragm joint was originally proposed and experimentally investigated by Tao et al. [18]. The six specimens used for FE model validation in the present study were obtained from the subsequent experimental program reported by Feng et al. [24], in which the axial compression ratio and FBP thickness were selected as the main test parameters.

Six interior specimens with identical overall dimensions and reinforcement layouts were used as benchmark specimens, as shown in Fig. 2, with their detailed parameters summarized in Table 1.

Each specimen consisted of a 2200 mm RC column with a 400 mm × 400 mm square section connected to a steel beam 3400 mm long and 320 mm deep. The beam flanges were 400 mm wide near the column face and 18 mm thick. According to the test variables, the specimens were divided into an axial compression ratio series and a bidirectional FBP series. In both series, whole column-section diaphragms extended from the beam flanges into the joint region, and the beam web was cut off at the column face. The FBPs used in the specimens were 175 mm wide, which facilitated stirrup arrangement and concrete casting in the core region. In the axial compression ratio series, the FBP thickness was fixed at 10 mm. In the bidirectional FBP series, additional FBPs parallel to the web were provided. The thickness of four FBPs in the bidirectional FBP series varied from 6 to 10 mm.

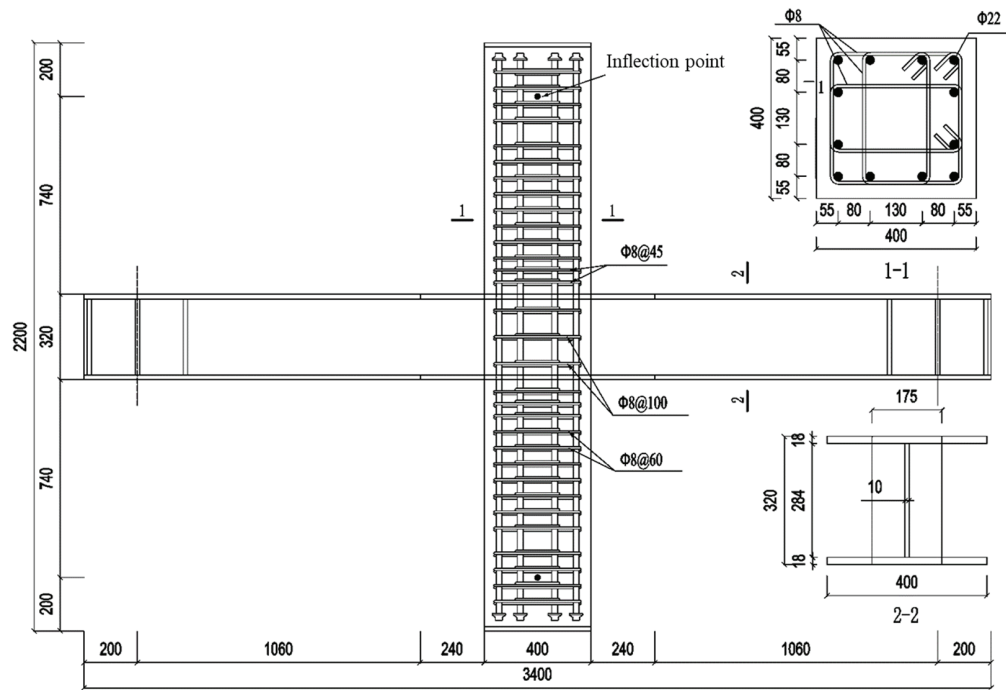


Figure 2: Dimensions and reinforcement details of specimens.

Table 1: Main parameters of specimens.

Specimen Group	Specimen	n	FBP Configuration	t/mm
Axial Compression Ratio Group	J-B-n0	0	Orthogonal FBP	10
	J-B-n22	0.22	Orthogonal FBP	10
	J-B-n38	0.38	Orthogonal FBP	10
FBP group	J-BB6	0.22	Orthogonal FBP; Parallel FBP	6
	J-BB8	0.22	Orthogonal FBP; Parallel FBP	8
	J-BB10	0.22	Orthogonal FBP; Parallel FBP	10

Note: n = axial compression ratio; t = thickness of the FBP.

The joint details are illustrated in Fig. 3. For the axial compression ratio series, the axial compression ratio varied from 0 to 0.38 while the joint detailing remained unchanged. For the bidirectional FBP series, the geometry and reinforcement were kept constant, and only the thickness of the FBPs was varied.

The experimental setup is shown in Fig. 4. The beam and column ends were pin-supported, and cyclic lateral displacement was applied at both beam ends under displacement control. The drift amplitudes ranged from 0.5% to 6.5%, as shown in Fig. 5. All specimens exhibited joint shear failure. The measured concrete and steel material strengths used in the FE models are summarized in Table 2. Because the experimental parameter range was limited, numerical analysis was subsequently used to extend the investigation.

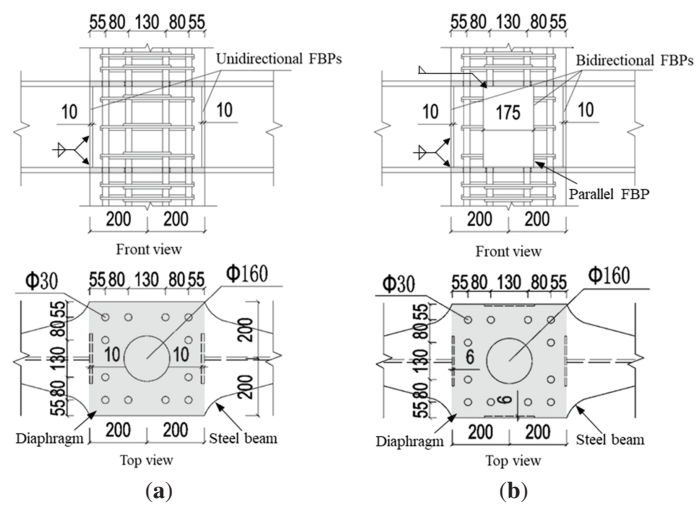


Figure 3: Design details of the joint. (a) Axial compression ratio group; (b) bidirectional FBP group.

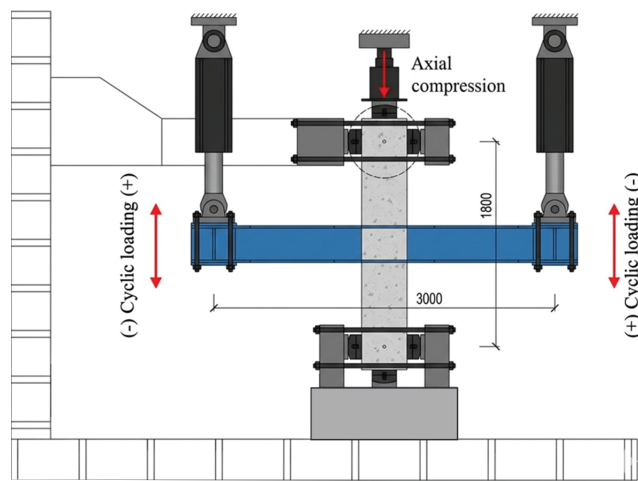


Figure 4: Test setup.

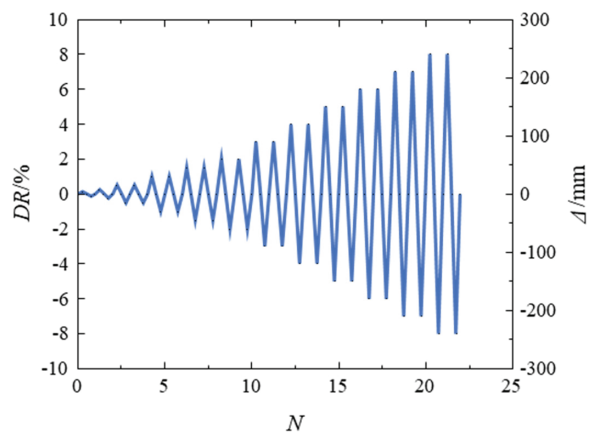


Figure 5: Loading protocol.

Table 2: Material strengths of concrete and steel.

Specimen Group	Concrete Strength	Steel Type	Application	f_y/MPa	f_u/MPa
J-B-n22 J-BB10	$f_{cu} = 56.0 \text{ MPa}, f'_c = 45.4 \text{ MPa}$	D8	Stirrup	484	634
		D22	Longitudinal bar	426	603
		t10	Beam web, FBP	361	516
		t18	Beam flange	356	559
J-B-n0 J-B-n38 J-BB6 J-BB8	$f_{cu} = 50.9 \text{ MPa}, f'_c = 41.2 \text{ MPa}$	D8	Stirrup	471	643
		D22	Longitudinal bar	414	604
		t6	FBP	490	596
		t8	FBP	409	556
		t10	Beam web, FBP	414	559
		t18	Beam flange	391	537

Note: The specimens were fabricated in two batches, resulting in differences in the material strength.

3 Numerical Simulation

3.1 Development of the Numerical Model

The nonlinear finite element program DIANA 10.3 was used to model six RCS joint specimens with whole column-section diaphragms. To improve computational efficiency, a half-model was established with respect to the joint mid-plane normal to the Y-direction, leveraging the geometric and loading symmetry of the specimens. Concrete and steel plates were modeled with HX24L eight-node solid elements. Longitudinal reinforcement was represented by beam elements, allowing the dowel action of the bars to be captured. Line to solid interface elements were introduced to simulate bond-slip between the longitudinal reinforcement and the surrounding concrete.

The transverse reinforcement was modeled using embedded truss elements fully coupled to the concrete. Surface-to-surface interface elements were employed at the steel-concrete contact surfaces to represent local interaction between the steel plates and the surrounding concrete.

The mesh pattern, boundary conditions, and loading protocol are shown in Fig. 6. A refined mesh was adopted in the joint core, where most of the inelastic response was expected. In contrast, a coarser mesh was used in the beam and column members. The element size was 20 mm in the joint region and 40 mm in the remaining beam and column regions. In the present DIANA model, the concrete tensile response was simulated using a fracture-energy-based formulation with crack band regularization to mitigate mesh dependence.

The boundary conditions of the model were defined as follows:

- along the centerline parallel to the Y-axis on the top end interface of the concrete column, the translational degrees of freedom in the X and Y directions were restrained;
- along the centerline parallel to the Y-axis on the bottom end interface of the concrete column, the translational degrees of freedom in the X, Y, and Z directions were restrained;
- on the symmetry plane perpendicular to the Y-axis, the translational degree of freedom in the Y direction was restrained for all nodes

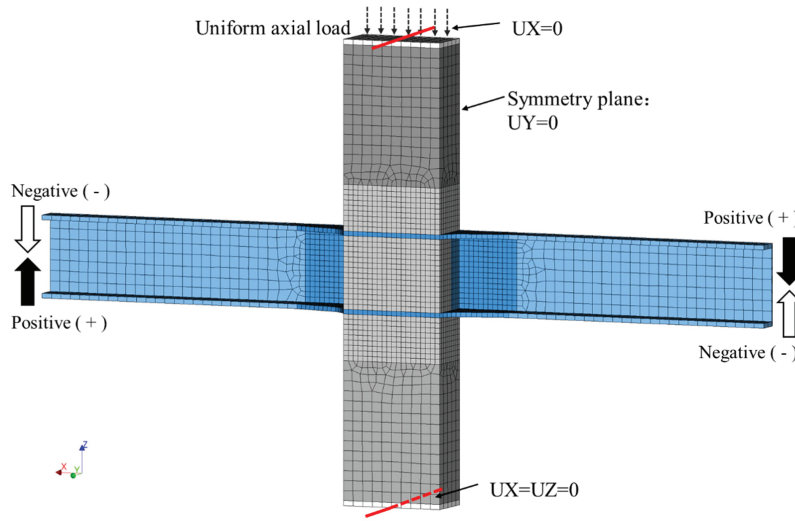


Figure 6: Diagram of the mesh, boundary conditions, and loading conditions of the FE model.

Hysteretic loading was applied under displacement control. Because the symmetry plane was defined normal to the Y-axis, Z-direction displacements were imposed at the center points of the webs at the two opposite beam ends, consistent with the experimental setup.

The analysis was performed with a step size of 1 mm until the target displacement was reached.

3.2 Material Modeling

The concrete was simulated using the Total Strain Rotating Crack model. The compressive constitutive behavior of the concrete was defined by the Maekawa cracked concrete curve [25], which accurately captures the cumulative damage under cyclic loading. The compressive stress-strain curve of this constitutive model is illustrated in Fig. 7a, and the governing equations are given as follows. Specifically, Eqs. (1)–(4) describe the loading branch, Eqs. (5) and (6) represent the unloading branch, and Eq. (7) defines the reloading branch:

$$\sigma = \omega K E_c (\varepsilon - \varepsilon_p) \quad (1)$$

$$K = \exp \left(-0.73 \frac{\varepsilon}{\varepsilon_c} \left(1 - \exp \left(-1.25 \frac{\varepsilon}{\varepsilon_c} \right) \right) \right) \quad (2)$$

$$\varepsilon_p = \left(\frac{\varepsilon}{\varepsilon_c} - \frac{20}{7} \left(1 - \exp \left(-0.35 \frac{\varepsilon}{\varepsilon_c} \right) \right) \right) \varepsilon_c \quad (3)$$

$$\varepsilon_c = 2.0 \frac{f'_c}{E_c} \quad (4)$$

$$\sigma = \omega K E_c (\varepsilon - \varepsilon_p) \alpha \quad (5)$$

$$\alpha = K^2 + \left(\frac{\sigma}{K E_c (\varepsilon_0 - \varepsilon_p)} - K^2 \right) \left(\frac{\varepsilon - \varepsilon_p}{\varepsilon_0 - \varepsilon_p} \right)^2 \quad (6)$$

$$\sigma = \omega \left(\sigma_{cmax} - (\sigma_{cmax} - \sigma_0) \frac{\varepsilon_{cmax} - \varepsilon}{\varepsilon_{cmax} - \varepsilon_0} \right) \quad (7)$$

where, σ and ε are the compressive stress and strain of concrete, respectively; ε_c is the strain corresponding to the peak compressive stress; ε_p is the plastic strain; σ_0 and ε_0 are the current compressive stress and strain; ω is the strength reduction factor induced by the orthogonal tensile strain; σ_{cmax} and ε_{cmax} are the maximum

stress and strain reached during the compression history; E_c is the elastic modulus of concrete; and K is the damage factor.

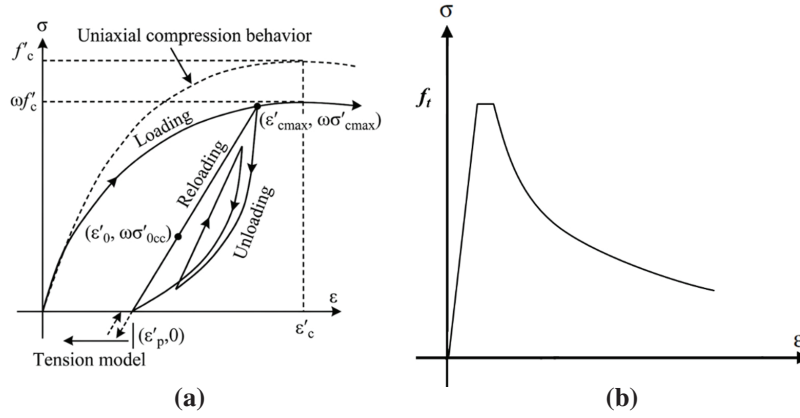


Figure 7: Constitutive relationships for concrete under compression and tension. (a) Maekawa cracked concrete; (b) JSCE tension stiffening.

The tensile stress-strain relationship of the concrete was defined using the JSCE tension stiffening model, as shown in Fig. 7b. The equation is expressed as follows:

$$\sigma = f_t (\epsilon_{tu}/\epsilon)^c \quad (8)$$

where, σ and ϵ denote the average tensile stress and strain of concrete, respectively; f_t is the uniaxial tensile strength; ϵ_{tu} is the cracking strain; and c is the tension-stiffening parameter, which is typically taken as 0.4 for deformed bars.

The constitutive behavior of all steel components was defined using the Von Mises isotropic hardening plasticity model. The uniaxial stress-strain relationship was characterized by a bilinear model, as given below:

$$\sigma = \begin{cases} E\epsilon, & \epsilon < \epsilon_y; \\ E'(\epsilon - \epsilon_y) + E\epsilon_y, & \epsilon \geq \epsilon_y. \end{cases} \quad (9)$$

where σ and ϵ denote the stress and strain of steel, respectively; ϵ_y is the yield strain; E is the initial elastic modulus; and E' is the post-yield tangent modulus, i.e., the slope of the plastic branch, which was taken as $0.01E$ to account for strain hardening.

Three-dimensional nonlinear interface elements were introduced at the interface between the steel beam and the concrete to simulate interfacial contact behavior. In this finite element simulation, a “hard contact” relationship was defined for the interface’s normal behavior. This formulation prevents interpenetration between steel and concrete elements under compression, while permitting separation and gap formation under tension. The tangential behavior of the interface was governed by the Coulomb friction model, as shown in Fig. 8. The adhesive bond was set to 0.4 MPa, and the interface friction angle was taken as 30° [26].

In typical beam-column joints, the bond-slip behavior significantly influences the overall hysteretic performance. To accurately capture the mechanical response of the specimens, the proposed finite element model explicitly accounts for the bond-slip effect between the column longitudinal reinforcement and the surrounding concrete. The tangential bond-slip relationship between the reinforcement and the concrete was defined using the bond stress-slip model proposed in CEB-fib 2010 [27], as shown in the figure below.

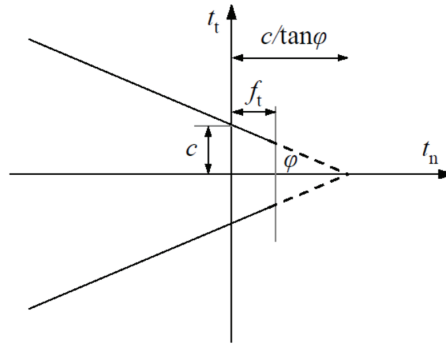


Figure 8: Coulomb friction model.

The bond stress-slip relationship illustrated in Fig. 9 can be formulated as:

$$\tau_b = \tau_{b \max} (s/s_1)^\alpha \quad 0 \leq s \leq s_1 \quad (10)$$

$$\tau_b = \tau_{b \max} \quad s_1 \leq s \leq s_2 \quad (11)$$

$$\tau_b = \tau_{b \max} - (\tau_{b \max} - \tau_{bf}) (s - s_2) / (s_3 - s_2) \quad s_2 \leq s \leq s_3 \quad (12)$$

$$\tau_b = \tau_{bf} \quad s_3 < s \quad (13)$$

where, α is a curve parameter taken as 0.4; s_1 and s_2 are 1 and 2 mm, respectively; $s_3 = c_{\text{clear}}$, where c_{clear} denotes the clear spacing between adjacent transverse ribs of the reinforcement; $\tau_{b \max}$ is the maximum bond stress, calculated as $2.5 (f'_c)^{0.5}$; and τ_{bf} is the residual bond stress, taken as 0.4 $\tau_{b \max}$.

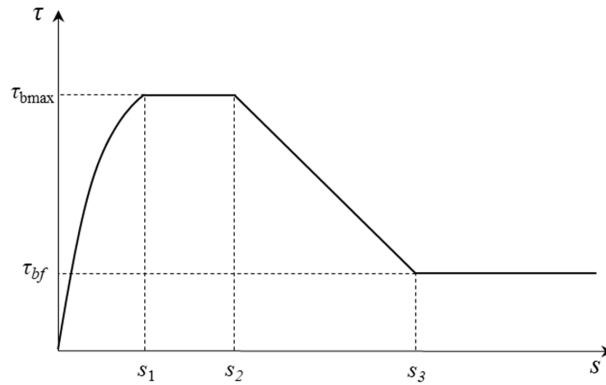


Figure 9: Bond-slip relationship between column bar and concrete.

3.3 Experimental Verification

Fig. 10 presents a comparison between the numerically simulated and experimentally measured load-displacement hysteretic curves for the whole column-section diaphragm RCS specimens under varying axial compression ratios and bearing plate thicknesses.

Overall, the numerical results exhibited good agreement with the experimental data. The simulated initial stiffness and peak bearing capacity closely matched the experimental observations. The corresponding quantitative comparison of initial stiffness, peak load, and displacement at peak load is summarized in Table 3. However, a slight discrepancy was observed in the unloading stiffness, which may be attributed to limitations in the unloading branch of the concrete constitutive model. In general, the simulated and

experimental hysteresis curves exhibited similar shapes. Notably, for specimen J-B-n0 subjected to zero axial load, both the simulated and experimental curves demonstrated a pronounced pinching effect, verifying that the proposed finite element model can reasonably capture the hysteretic behavior of the specimens.

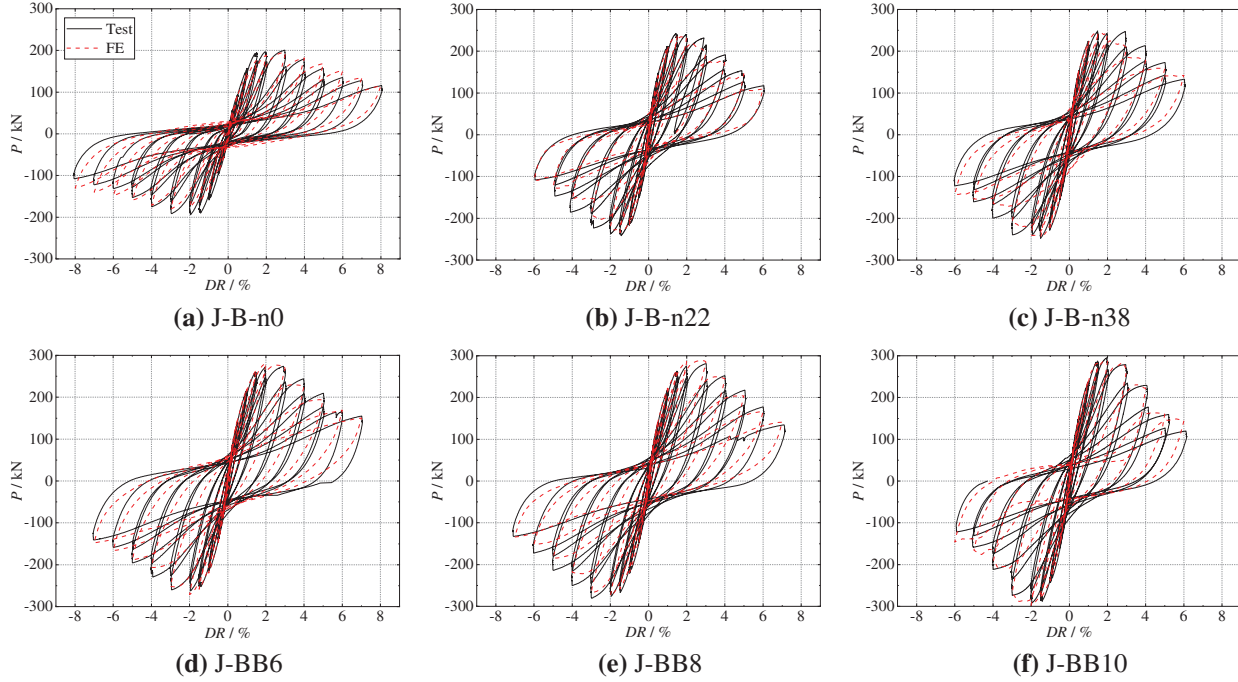


Figure 10: Comparison of experimental and simulated hysteretic curves.

Table 3: Quantitative comparison between FE and test results.

Specimen	K_0^{test} (kN/mm)	K_0^{FE} (kN/mm)	K_0 Error (%)	P^{test} (kN)	P^{FE} (kN)	P Error (%)	Δ^{test} (mm)	Δ^{FE} (mm)	Δ Error (%)
J-B-n0	16.48	15.48	-6.02	197.17	187.26	-5.02	36.92	44.50	20.52
J-B-n22	25.64	23.19	-9.55	242.24	234.55	-3.17	21.55	22.5	4.41
J-B-n38	25.19	24.33	-3.44	248.37	245.23	-1.26	22.48	21.75	-3.25
J-BB6	23.24	23.46	0.95	268.14	276.49	3.11	36.50	30.00	-17.81
J-BB8	23.98	23.10	-3.68	280.76	282.30	0.55	44.73	35.00	-21.76
J-BB10	26.09	23.68	-9.22	293.71	293.81	0.04	29.40	29.25	-0.5

3.4 Crack Development and Failure Mechanism

The numerical results for specimens J-B-n22 and J-BB10 which represented two different joint details, were selected to illustrate the predicted cracking process and failure mechanism. The principal tensile strain contours indicated that specimen J-B-n22 failed in joint shear, in good agreement with the experimental observations. Based on the loading history, the evolution of cracking under cyclic loading was compared with the observed crack patterns.

Three key stages were identified during the loading process: (1) crack initiation; (2) attainment of the peak load-carrying capacity; and (3) degradation of the load to 50% of the peak value, at which the specimen was regarded as failed. This definition is consistent with the failure criterion adopted in the experiments. The peak load-carrying capacity was reached at a drift ratio of 1.5%.

As shown in Fig. 11, a detailed comparison between the numerical strain contours and the experimental crack patterns revealed that the cracks were primarily concentrated near the diagonals of the joint core. During hysteretic loading, cracks propagated across the joint in both the positive and negative loading directions, forming an X-shaped pattern. The strain distribution in the FE model agreed well with these experimental crack patterns, with the maximum principal tensile strains observed along the diagonal directions. A further comparison of the crack pattern and strain distribution in the column concrete indicated that some flexural cracks developed during the test, and this phenomenon was also captured by the finite element results, which showed representative high-strain zones at the column ends due to the significant bending moments sustained in these regions.

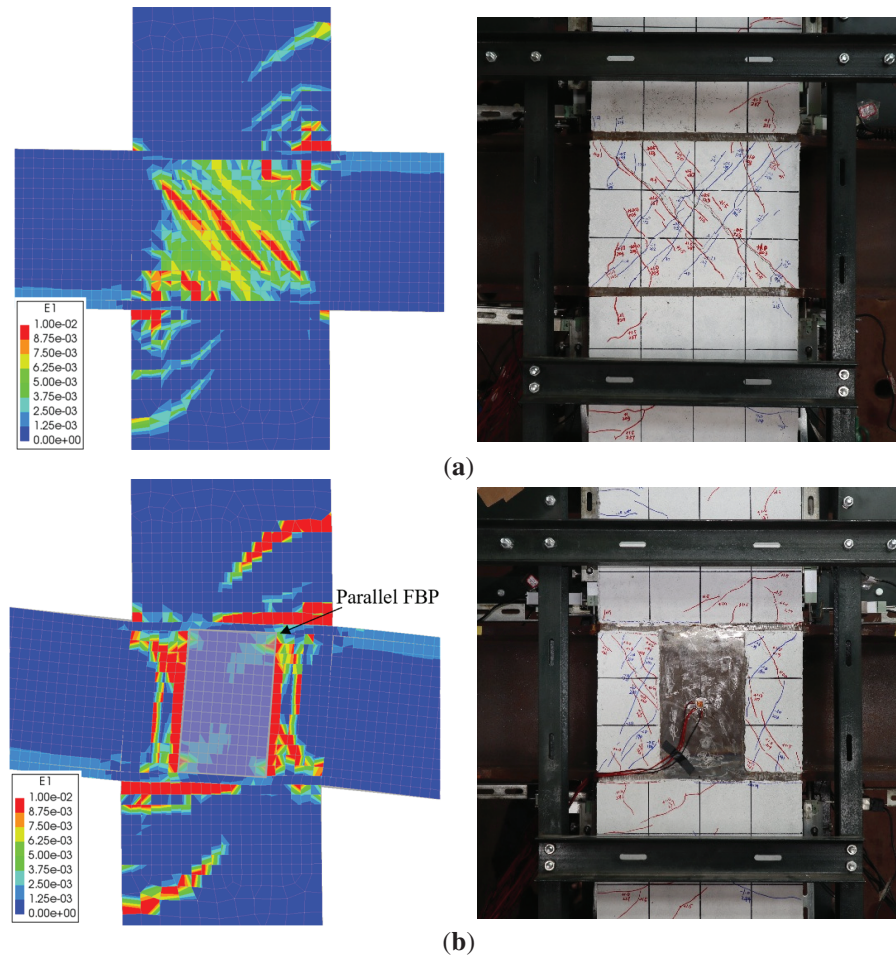


Figure 11: Comparison of experimental and numerical surface crack patterns at peak load. (a) J-B-n22; (b) J-BB10.

Fig. 12 illustrates the evolution of the principal tensile strain contours for specimen J-B-n22 at various displacement levels. During the initial loading stage, inclined cracks first emerged at the center of the joint core, which is consistent with the initial crack distribution observed in the experiment. As the displacement load increased, these diagonal cracks gradually extended from the center of the specimen toward the diagonal

corners. The number of diagonal cracks also increased progressively, dividing the entire joint region into several slender concrete blocks.

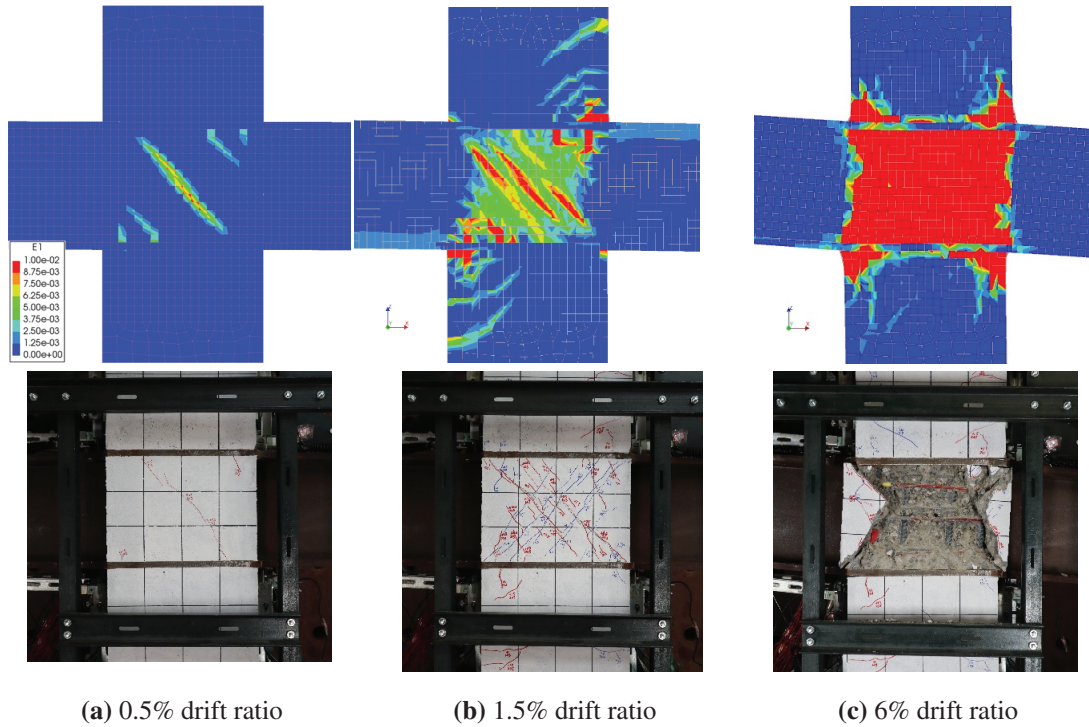


Figure 12: Comparison of experimental and numerical crack patterns for specimen J-B-n22.

In the later stages of loading, further increases in displacement led to a reduction in the joint's bearing capacity. The principal tensile strain contours show that almost the entire joint region became a high-strain cracked zone. This corresponded well with the experimental observations, where the joint concrete experienced severe cracking, crushing, and spalling, ultimately leading to the loss of the concrete cover for both stirrups and longitudinal rebars. Consequently, the load-bearing capacity of the specimen decreased gradually due to severe concrete crushing.

The equivalent stress contours of the concrete are shown in Figs. 13 and 14. The high-stress regions within the joint core were concentrated along the diagonal compression strut, indicating that the concrete diagonal strut mechanism is a crucial component of the shear resistance. By comparing the equivalent stress contours on the outer concrete surface at different drift ratios, it is evident that the equivalent stress increases progressively with the displacement load. Furthermore, the geometric shape of the compression zone transitions from a parallelogram to a drum-like shape.

A comparison of the concrete equivalent stress contours at the middle plane under various drift ratios revealed that, as the drift ratio increases from 1% to 2%, the equivalent stress of the concrete increases accordingly. However, within the drift ratio range of 2% to 3%, the equivalent stress values gradually declined. This reduction is accompanied by a narrowing of the diagonal compression strut, resulting in a decrease in the joint's load-carrying capacity. Furthermore, when comparing the concrete stress states at the identical drift ratio between the exterior surface and the middle plane, it was evident that the diagonal compression strut is wider at the middle plane than on the exterior surface. This phenomenon was attributed to the presence of

the orthogonal face bearing plate, which sustains a portion of the compressive stress and effectively extends the diagonal compression strut.

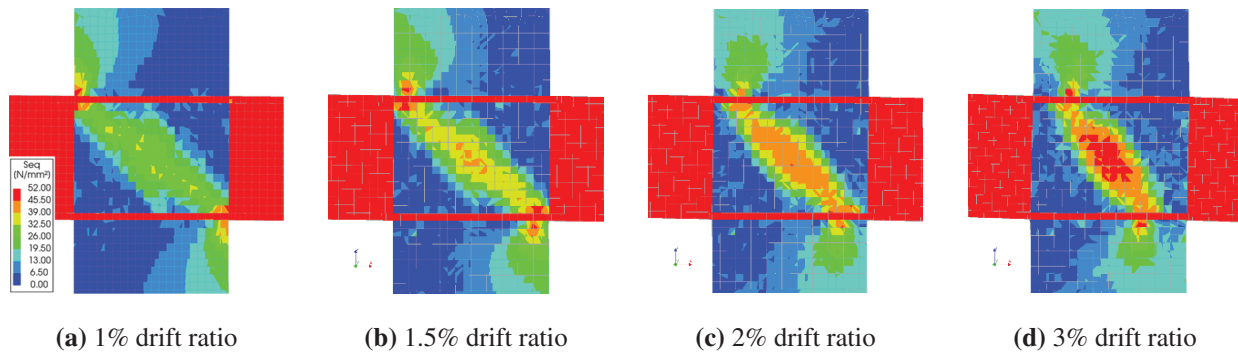


Figure 13: Equivalent stress contours on the outer surface of specimen J-B-n22.

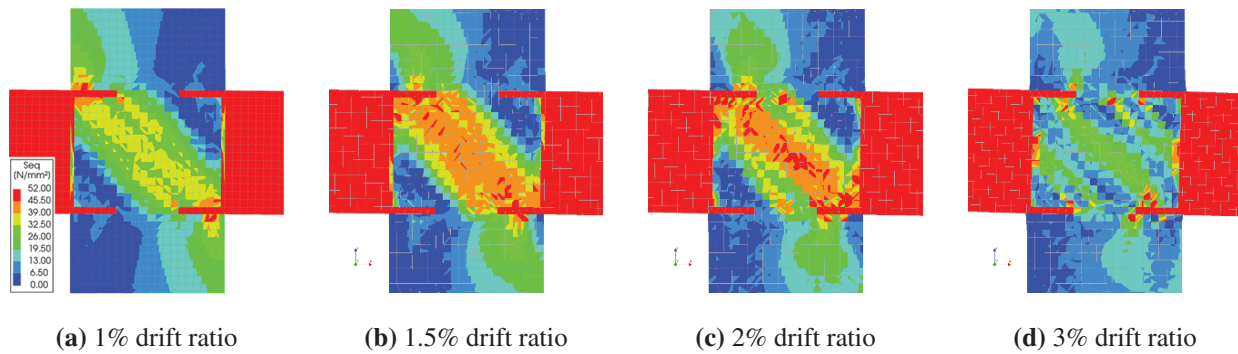


Figure 14: Equivalent stress contours on the mid-plane of specimen J-B-n22.

3.5 Parametric Study

In this section, a comprehensive parametric study was conducted using the validated FE model to investigate the effects of two key experimental parameters: the axial compression ratio and the thickness of the face bearing plate. This analysis expanded the range of parameter values, thereby serving as a valuable supplement to the experimental data.

The FE results for varying axial compression ratios are shown in Fig. 15a. The axial compression ratio ranged from 0 to 0.6, with increments of 0.1. As illustrated in the figure, the axial compression ratio significantly influenced the seismic performance of the joints. An increase in the axial compression ratio enhanced both the initial stiffness and the load-bearing capacity. Specifically, within the range of 0 to 0.3, a higher axial load improved the bearing capacity while maintaining relatively good ductility. However, when the axial compression ratio exceeded 0.4, the peak bearing capacity remained largely unchanged, whereas the displacement corresponding to the peak load decreased. Furthermore, the descending branch of the skeleton curve became steeper, indicating premature joint failure and reduced overall ductility.

The skeleton curves of the joints with different FBP thicknesses are shown in Fig. 15b. It was evident that the plate thickness had a certain influence on the joint response. At small drift ratios, the six skeleton curves almost overlapped, indicating that the plate thickness has a limited effect on the initial stiffness. The curves diverged at a drift ratio of 2%, indicating that a thicker face-bearing plate resulted in higher bearing capacity. Nevertheless, this enhancement in the peak load was relatively marginal. Increasing plate thickness

from 6 to 16 mm resulted in only an approximately 10% increase in the peak load. This is attributed to the fact that the thicker FBPs did not fully yield over their entire section. Therefore, the additional steel area was not fully mobilized, and the joint resistance remained mainly governed by the concrete-dominated diagonal strut mechanism.

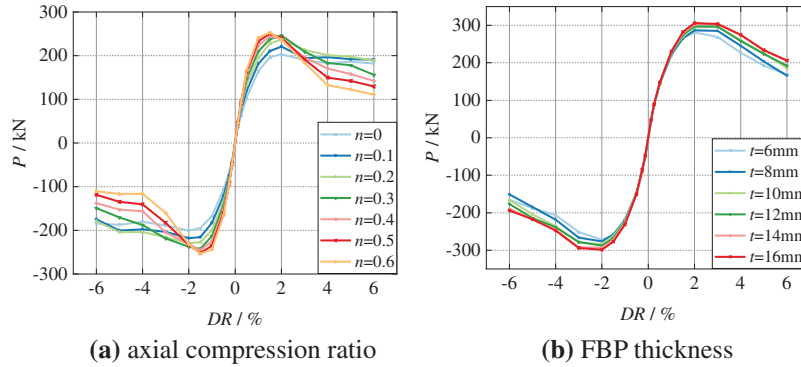


Figure 15: Results of the finite element parametric study.

3.6 Force-Transfer Mechanism and Joint Shear Resistance

In reinforced concrete joints, shear transfer within the joint core region is generally attributed to several mechanisms, including diagonal strut action, truss action, shear friction, block shear, and beam shear. For RCS joints, existing design models commonly evaluate the concrete's contribution to shear capacity based on the diagonal compression strut mechanism.

Under lateral loading, the concrete in the joint core is subjected to vertical compression from the column and horizontal compression from the FBPs. The interaction of these orthogonal forces generates a diagonal compression field as shown in Fig. 16. Within the beam-flange width, the concrete is further confined by the flanges, FBPs, and transverse reinforcement, resulting in an approximately triaxial stress state and enhanced shear resistance.

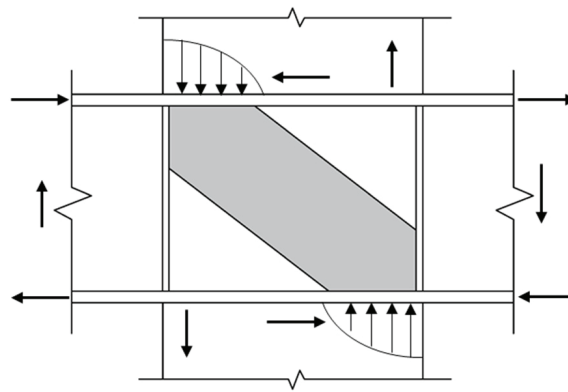


Figure 16: Concrete diagonal compression strut mechanism.

In contrast to conventional RC joints, most RCS joints contain a steel web within the core, which contributes to shear transfer through a “steel frame-shear wall” mechanism as shown in Fig. 17. In the joint detail considered in this study, the web is interrupted at column face to improve constructability. However,

in specimens with parallel FBPs, these plates play a role similar to that of a continuous steel web, thereby actively contributing to joint shear resistance.

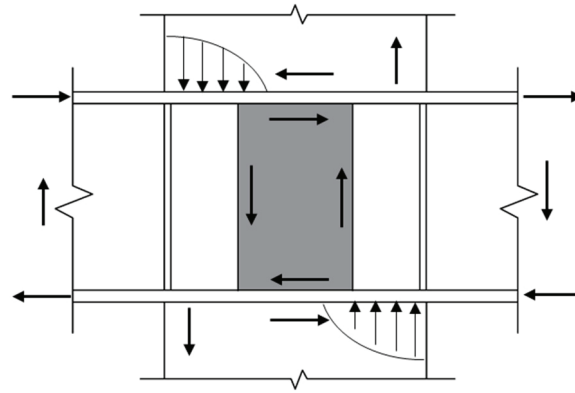


Figure 17: Frame-shear wall mechanism of parallel face bearing plates.

As shown in Fig. 18, the transverse stirrups contribute to the joint shear capacity primarily through the truss mechanism, with three main roles: confining the core concrete, directly resisting shear, and preventing the longitudinal reinforcement from buckling in compression. Following concrete cracking, the strains in the stirrups increase markedly, indicating their effectiveness in restraining the subsequent shear deformation of the joint core. In specimens with parallel FBPs, a portion of the shear demand is transferred to the steel plates, thereby delaying the development of strain in the stirrups. Because the stirrups within the joint region typically do not reach full yield prior to joint failure, most design equations apply a reduction factor to adjust their calculated contribution to the overall shear strength.

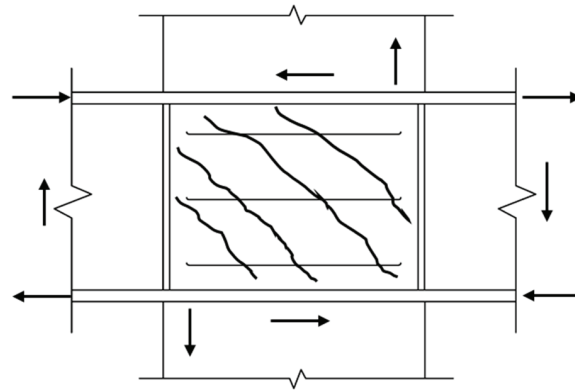


Figure 18: Truss mechanism of stirrups.

4 Joint Shear Capacity Analysis

Three representative design provisions were examined to evaluate the shear resistance of the proposed RCS joints: the ASCE guidelines [28], the AIJ specification [29], and the CECS 347:2013 technical specification [30].

The primary difference among these methods lies in their treatment of the concrete's contribution to the overall shear capacity. The ASCE approach explicitly distinguishes between the inner and outer concrete

elements based on varying degrees of confinement. In contrast, the AIJ and CECS formulations evaluate the joint core more holistically and then add the steel-related contributions.

4.1 Existing Shear-Strength Equations

Within the ASCE framework, joint shear resistance is attributed to web action coupled with concrete diagonal compression mechanisms. Since the proposed whole column-section diaphragm joint does not contain a continuous web passing through the joint core, the evaluation adopted herein focuses on the concrete and stirrup contributions:

$$V_j = 1.7\sqrt{f'_c}b_i h_c + 0.4\sqrt{f'_c}b_o h_c + \frac{0.9A_{sh}f_y h_c}{s_h} \quad (14)$$

where f'_c is the cylinder compressive strength of concrete; b_i and b_o are the effective widths of the inner and outer concrete regions; h_c is the column depth; A_{sh} is the stirrup area within one spacing; s_h is the stirrup spacing; and f_y is the stirrup yield strength.

The AIJ recommendation, which was calibrated against a comprehensive experimental database, formulates the joint shear strength as the sum of the contributions from the steel web, cover plates, transverse reinforcement, and concrete:

$$V_j = V_w + V_f + V_h + V_c \quad (15)$$

$$V_w = C_1 A_w f_{wy} / \sqrt{3} \quad (16)$$

$$V_f = 0.5 A_f f_{fy} / \sqrt{3} \quad (17)$$

$$V_h = 0.25 p_w f_y b_c d \quad (18)$$

$$V_c = 0.04 C_2 C_3 b_c h_c f'_c \delta \quad (19)$$

where V_w , V_f , V_h , and V_c denote the contributions of the web, cover plates, transverse reinforcement, and concrete, respectively; A_w and A_f are the sectional areas of the web and cover plates; f_{fy} is the yield strength of the cover plates; p_w is the transverse reinforcement ratio; b_c is the column width; d is the internal lever arm between the tension and compression bars; δ is the joint shape factor; and C_1 , C_2 , and C_3 are detailing coefficients given in Ref. [29].

For comparative analysis, CECS 347:2013 also provides a compact expression for the shear strength of composite RCS joints:

$$V_j = 0.15 \alpha b_c h_c f_c + p_w f_y b_c h_{b0} + 0.4 \sum A_f f_{fy} \quad (20)$$

where α is the joint location factor, equal to 1.0 for interior joints; f_c is the compressive strength of concrete; and h_{b0} is the clear beam depth.

4.2 Comparison of Prediction Results

Fig. 19 presents the comparison of calculated shear strength from three methods. A notable limitation of all three methods is that the axial compression ratio is not explicitly considered. Consequently, specimens featuring identical structural detailing but subjected to varying axial loads yield identical theoretical predictions.

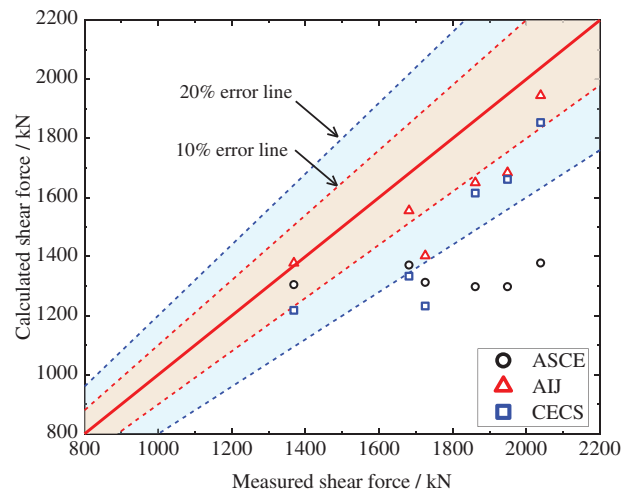


Figure 19: Comparison of calculated shear strength from three formulas.

Table 4 shows the comparison of calculated-to-experimental ratios for different design formulas. Among the evaluated methods, the ASCE method yielded the lowest predictions for the proposed specimens. This is mainly because it was derived for conventional through-beam joints. Therefore, the ASCE method did not directly capture the force-transfer characteristics associated with the whole column-section diaphragm and the parallel FBPs. In addition, restricting the effective concrete width strictly to the FBP width further reduced the contribution from the concrete term in the shear strength equation. In contrast, the AIJ and CECS equations provided closer estimates because their theoretical idealizations better aligned with the overall behavior of the proposed joint, in which the concrete acts as an integral component. Nevertheless, these methods provided conservative estimates because they fail to account for the beneficial confinement effects of axial compression.

Table 4: Comparison of calculated-to-experimental ratios for different design formulas.

Formula	J-B-n0	J-B-n22	J-B-n38	J-BB6	J-BB8	J-BB10	Mean	SD
ASCE	0.953	0.815	0.761	0.697	0.666	0.676	0.761	0.110
AIJ	1.007	0.925	0.813	0.886	0.864	0.954	0.908	0.069
CECS347:2013	0.890	0.793	0.714	0.867	0.852	0.909	0.837	0.072

4.3 Proposed Modification to the CECS-Based Method

The CECS 347:2013 formulation was selected as the baseline for modification because it treats the joint core concrete as an integral shear-resisting component, which is broadly consistent with the force-transfer mechanism of the whole column-section diaphragm joint considered in this study. Compared with the AIJ expression, the CECS equation is also more concise and involves fewer detail-dependent coefficients, making it more convenient for calibration and practical engineering applications. To accurately evaluate the proposed joint, it's necessary for this standard formulation to be modified, as its original form does not account for the specific confinement provided by the whole column-section diaphragm or the beneficial effects of axial compression.

Both the experimental observations and FE results indicate that core concrete resists the dominant portion of the joint shear force. As displacement increases, diagonal cracking develops in the core and gradually forms a compression strut across the joint. On this basis, the proposed modification focuses on reformulating the concrete contribution term using a diagonal-strut representation, while retaining the original treatment of the stirrup and parallel FBP contributions.

Accordingly, the concrete contribution can be expressed as follows:

$$V_c = B_e b_j f_c \cos \theta \quad (21)$$

where B_e is the equivalent depth of the diagonal concrete strut, b_j is the joint width, f_c is the axial compressive strength of concrete, and θ is the strut inclination.

Using the conventional diagonal-strut idealization, B_e may be related to the diagonal dimension of the joint core [31]:

$$B_e = \alpha \sqrt{h_c^2 + h_b^2} \quad (22)$$

where α is a coefficient identified from test data; h_c is the column depth; and h_b is the beam depth. Introducing $h_b = \beta h_c = \beta h_j$ leads to the following simplified form:

$$B_e = \alpha \sqrt{h_j^2 + \beta^2 h_j^2} = \alpha \sqrt{1 + \beta^2} h_j \quad (23)$$

By defining $\gamma = \alpha \sqrt{1 + \beta^2} \cos \theta$, the expression for the concrete shear strength can be rewritten as:

$$V_c = B_e b_j f_c \cos \theta = f_c b_j \alpha \sqrt{1 + \beta^2} h_j \cos \theta = \gamma f_c b_j h_j \quad (24)$$

where γ is defined here as the nominal shear-compression ratio, representing the combined effect of local confinement and force transfer provided by the diaphragm, FBPs, and surrounding reinforcement.

For the proposed joint configuration, the coefficient γ was determined by back-calculating the concrete shear contribution from the measured joint shear strength after deducting the calculated contributions of the parallel FBPs and the joint stirrups based on the measured material strengths:

$$\gamma = \frac{V_j^t - V_p - V_s}{f_c b_j h_j} \quad (25)$$

where V_j is the measured joint shear force at peak resistance, V_p and V_s are the calculated contributions of the parallel FBPs and stirrups, respectively, and b_j and h_j are the width and depth of the joint section.

Since the experimental database was limited, the FE parametric study was used to broaden the calibration range. Specifically, the axial compression ratio was expanded to span from 0 to 0.6, and the FBP thickness range was extended to 6–16 mm. The resulting trends in coefficient γ with respect to the axial compression ratio and the FBP thickness are presented in Figs. 20 and 21, respectively.

For the axial compression ratio series, all geometric and structural detailing parameters remained constant, with only the axial compression ratio being varied. Regression analysis of the γ - n relationship, together with the FE results and the requirement for a practical design expression, yielded the following empirical relationship:

$$\gamma = \begin{cases} 0.18 + 0.18n & 0 \leq n \leq 0.4 \\ 0.252 & 0.4 \leq n \leq 0.6 \end{cases} \quad (26)$$

The corresponding concrete contribution can then be evaluated as:

$$V_c = \gamma f_c b_j h_j = \begin{cases} (0.18 + 0.18n) f_c b_j h_j & 0 \leq n \leq 0.4 \\ 0.252 f_c b_j h_j & 0.4 \leq n \leq 0.6 \end{cases} \quad (27)$$

For the FBP thickness series, the coefficient γ exhibited no clear dependence on the plate thickness. Consequently, the effect of the FBP thickness on the concrete shear contribution was neglected in the proposed modification, and Eq. (27) was applied to both specimen groups.

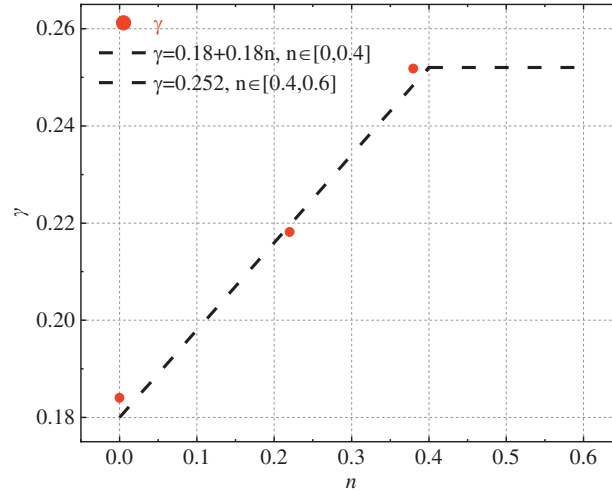


Figure 20: Relationship between γ and n for the axial compression ratio series.

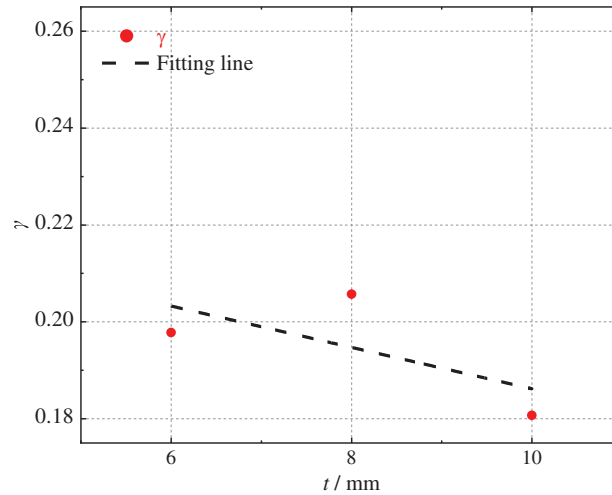


Figure 21: Relationship between γ and t for the FBP thickness series.

Accordingly, the shear strength of the proposed joint is taken as the sum of the concrete, stirrup, and parallel-FBP contributions, with the concrete term modified to account for the effect of axial compression under whole column-section diaphragm detailing:

$$V_j = V_c + V_s + V_p = \gamma f_c b_j h_j + p_w f_y b_c h_c + 0.4 \sum A_f f_{fy} \quad (28)$$

The predictions obtained from the revised equation are shown in Fig. 22 below:

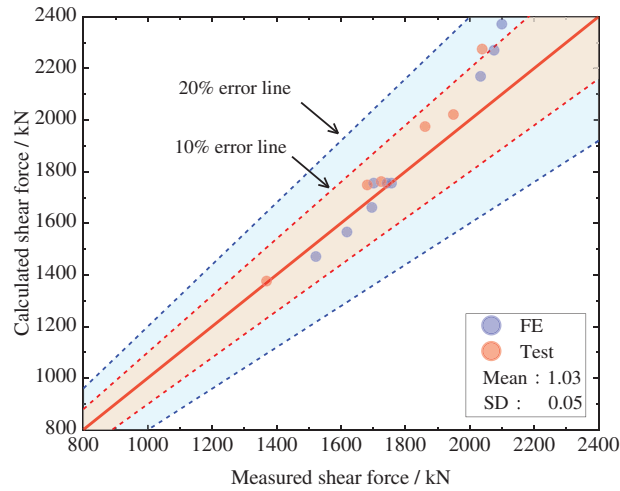


Figure 22: Comparison between calculated and measured joint shear force.

In comparison to the original CECS equation, the revised formulation showed better agreement with the combined experimental and FE results. For the majority of the evaluated specimens, the prediction error was contained within approximately 10%. This indicated that the proposed expression offers a more suitable and practical estimate of the shear capacity for the whole column-section type joint. It should be noted that the proposed expression represents a simplified, design-oriented modification of the CECS formulation. The shear contributions of the transverse reinforcement and steel plates are kept unchanged, and the concrete contribution is modified to account for the effect of the axial compression ratio. Therefore, the proposed equation should be regarded as a practical extension of the CECS method for the joint configuration investigated in this study.

5 Conclusions

Based on finite element simulations, parametric analyses, and evaluation of current design equations, the main conclusions of this study are summarized as follows:

1. The proposed FE model can reproduce both the global and local behavior of RCS joints with whole column-section diaphragms with satisfactory accuracy. In particular, the model successfully captured the hysteretic response, the joint shear failure mode, and the development of diagonal cracks observed in the tests.
2. The whole column-section diaphragm detail effectively suppressed brittle local bearing failure and promoted a stable joint shear response. Increasing the axial compression ratio from 0–0.3 improved the initial stiffness and peak strength of the joint, but excessive axial compression >0.3 reduced deformation capacity. For the proposed joint configuration, an axial compression ratio below 0.3 was recommended to maintain stable behavior.
3. The parallel FBPs contributed directly to shear transfer and increased joint strength. However, within the investigated range, increasing the parallel FBP thickness from 6 to 16 mm yielded a limited strength enhancement, because the FBPs did not fully yield over the section and the joint resistance remained governed by the concrete diagonal strut mechanism.
4. By modifying the concrete contribution term in the CECS formulation to account for the effect of axial compression ratio, the prediction accuracy of joint shear strength was improved. The revised expression

provided good agreement with the combined experimental and FE database. Therefore, it's suitable for preliminary design assessment of this novel joint configuration.

These recommendations are specifically applicable to RCS joints with whole column-section diaphragms within the evaluated ranges of material strengths and geometric proportions. Extrapolating these conclusions to significantly different joint typologies, ultra-high-strength materials should be approached with caution and require further experimental and numerical validation.

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