

Article

Experimental and Numerical Investigations of Seismic Performance of Prefabricated SRC Frame in Multi-Floored Grain Warehouse

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Abstract

As an innovative structural system aligned with construction industrialization, prefabricated Steel-Reinforced Concrete (SRC) structures are characterized by high load-bearing capacity, efficient material utilization, and rapid construction. In this study, the mechanical behavior, failure mechanisms, and ductility characteristics of a prefabricated SRC multi-floored grain warehouse frame were investigated through quasi-static cyclic loading tests. To complement the experimental program, high-fidelity numerical models were developed using Abaqus, incorporating concrete plastic damage and steel material non-linearity. The simulation results were rigorously validated against the experimental data. The findings indicate that the specimens exhibited typical shear failure modes with full hysteretic loops, demonstrating substantial energy dissipation capacity (equivalent viscous damping coefficient of 0.261). Notably, the results of the parametric study indicate that the integration of wall panels can significantly increase the load-bearing capacity and lateral stiffness of the frame system. The ductility of the samples was excellent, with displacement ductility coefficients ranging from 3.1 to 3.7. The ultimate inter-story drift angles at failure (1/49–1/38) substantially exceeded the code-specified limit (1/50), indicating robust collapse-prevention capacity. The numerical results strongly agreed with the experimental observations in terms of the hysteretic behavior, failure patterns, and skeleton curves, confirming the reliability of the modeling strategy for subsequent seismic performance analyses and parametric evaluations.

Keywords: prefabricated building; steel-reinforced concrete structure; multi-floor grain warehouse; seismic performance

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1. Introduction

Grain storage infrastructure is fundamental to ensuring national food security and maintaining stable strategic reserves [1]. With the rapid progression of urbanization and increasingly limited land resources, Multi-floored Grain Warehouses (MGW) have emerged as a major storage typology in large-scale logistics parks. Compared with traditional flat warehouses, MGW provides substantial advantages in terms of land use efficiency, mechanical automation, and storage performance [2–4]. These structures are gen-

erally characterized as heavy-duty systems, with large spans, high story heights, and extreme live loads—often 3 to 5 times those of conventional residential frames—to accommodate bulk grain storage capacities of up to 100,000 tons [5].

Traditional reinforced concrete (RC) frames used in MGWs often require excessively large cross-sections to satisfy load-bearing requirements, resulting in intensive on-site construction and associated environmental concerns. To overcome these limitations, Steel-Reinforced Concrete (SRC) structures have been introduced. By embedding steel sections within concrete members, compared with conventional RC members, SRC systems achieve superior ductility [6–8], enhanced energy dissipation capacity, and a reduction in cross-sectional area of more than 20% [9–12]. Furthermore, the construction industry is shifting toward prefabricated SRC systems to minimize onsite wet work, reduce environmental pollution, and shorten construction periods. At present, research on SRC structures focuses mainly on the seismic performance and strength of joints and frames. Im et al. [13] experimentally evaluated the seismic response of monolithic precast beam-column connections incorporating beam-end U-shaped keyways. Through cyclic loading tests on six full-scale specimens, the effects of the beam-end reinforcement ratio, seating length, and supplementary angle-steel configurations on the structural response were systematically examined. The results demonstrated that these precast joints exhibited seismic performance comparable to that of conventional monolithic concrete connections, with all the mechanical parameters satisfying the requirements of current design specifications. Parastesh et al. [14] developed an optimized ductile flexural joint for high-intensity seismic regions. Their findings revealed that the failure mechanism, characterized by flexural cracking within the beam-end plastic hinge, strictly followed the “strong column-weak beam” design philosophy. The incorporation of diagonal reinforcement within the joint core significantly delayed the initiation of diagonal cracking and reduced shear damage, resulting in a 30% increase in energy dissipation capacity compared with monolithic cast-in-place joints while maintaining comparable strength and ductility. Furthermore, Eom et al. [15] improved U-shaped keyway joints by applying a plastic-hinge relocation strategy. Through optimized anchorage and the use of irregular bar sections to adjust the beam-end reinforcement, bar slip and core shear failure were effectively mitigated, resulting in better mechanical performance than that of conventional monolithic connections. Complementing these studies, Rong et al. [16] demonstrated that the encasement of steel sections in SRC beam-column joints substantially increases global structural stiffness and ductility, thereby effectively improving overall seismic resistance. Xue et al. [17] conducted quasi-static tests on a 1:4-scale, six-story precast frame comprising cast-in-place columns and precast composite beams. Their findings indicated that the system exhibited favorable energy dissipation capacity and ductility. Damage developed through the sequential formation of plastic hinges—first in the beams and subsequently in the columns—which is consistent with the “strong column-weak beam” strategy, thereby verifying the seismic reliability of the hybrid precast structure. Although current research has begun to investigate the dynamic characteristics of prefabricated SRC structures, a substantial knowledge gap remains regarding their seismic performance under different grain loading scenarios. In particular, the synergistic mechanical interaction and load-transfer mechanisms between prefabricated SRC joints and wall panels under the combined effects of seismic action and out-of-plane grain pressure remain insufficiently understood.

The novelty of this study lies in the examination of the seismic response of prefabricated SRC grain-warehouse frames under the combined effects of wall-panel interaction and out-of-plane grain pressure, which has rarely been addressed in previous studies. To address these issues, a systematic investigation of prefabricated SRC multi-floored grain warehouses was conducted. By combining large-scale quasi-static testing with high-fidel-

ity finite element (FE) analysis, the mechanical behavior, failure patterns, and seismic performance of the specimens were rigorously evaluated. Particular attention was given to the structural response under the combined effects of material storage loads and seismic forces, thereby providing a theoretical basis and technical reference for practical engineering design and seismic resilience assessment.

2. Experimental Program

2.1. Construction of the Proposed Precast Frame

A multi-floored grain warehouse located in Guangdong was selected as the prototype, and a 1/2-scale model was designed for the experimental investigation. Q345-grade steel was used for the steel sections of both the beams and columns, with a uniform cross-section of $H220 \times 100 \times 8 \times 10$. The reinforcing bars and concrete consisted of HRB400-grade steel and C45 commercial concrete, respectively. The prefabricated steel-reinforced concrete (SRC) column was 3300 mm high and had a square cross-section of 400×400 mm. The longitudinal reinforcement comprised 12 ϕ 14 bars (corresponding to a reinforcement ratio of 4.6%), and ϕ 8 stirrups were installed at a constant spacing of 150 mm (ϕ 8@150).

The construction and assembly of the prefabricated SRC frame proposed in this study were divided into four stages: (1) prefabrication of structural components, (2) assembly of prefabricated beams and columns, (3) installation of sandwich wall panels, and (4) post-casting of columns, beams, and wall-panel joints. During the precasting of the foundation beams, the embedded steel sections within the foundation were bolted to the column steel sections to ensure reliable connectivity. At the cast-in-place column-foundation beam interface, the longitudinal reinforcement was connected using lap splices. A lap length of 500 mm was adopted, with the splices arranged in a staggered manner at intervals of 500 mm. At the beam-column joints, the top longitudinal bars of the beams were anchored using 90° hooks. These bars extended beyond the inner side of the exterior column reinforcement into the joint core. The horizontal projection length (including the bend) was maintained at no less than 0.4 l_{ab} , while the vertical extension length was at least 15d. The wall system consisted of 250 mm-thick prefabricated panels, comprising two 100 mm-thick concrete layers and a 50 mm intermediate insulation layer. To ensure structural integrity between the wall and frame, U-shaped hairpins (ring bars) were embedded along the panel edges and lapped with the longitudinal reinforcement of the columns. Quality control of the sample focused mainly on the prefabricated joints and post-cast connection regions. During fabrication, the positions of the steel sections, reinforcement cages, lap splices, and embedded U-shaped hairpins were inspected before concrete casting. The post-cast regions at the beam-column joints and wall-column interfaces were carefully compacted to ensure reliable force transfer between the prefabricated components and cast-in-place concrete. The concrete specimens were cured under standard conditions for 28 days, and material tests were performed before the structural tests. Potential construction defects, including insufficient concrete compaction, poor interface bonding, bolt loosening, and reinforcement misalignment, may reduce joint stiffness and accelerate the development of local damage. Therefore, these defects should be strictly controlled in practical engineering applications.

2.2. Material Properties

The prefabricated components were cast using C45-grade ordinary concrete, whereas the post-cast regions (wet joints) were constructed using C45-grade micro-expansive concrete. The specimens were subsequently cured under standard laboratory conditions for 28 days. Concurrently, three sets of concrete cubes measuring $150 \times 150 \times 150$ mm were prepared for each type of concrete and cured under the same conditions as the main spec-

imens. The compressive strength of the concrete cubes was measured using a TYA-2000 electrohydraulic compression testing machine in accordance with the relevant specifications [18]. The measured mechanical properties are summarized in Table 1. In addition, tensile tests on the steel sections and reinforcing bars were conducted using a DNS300 electronic universal testing machine, following the procedures specified in GB/T 228.1–2010 [19]. The detailed material parameters are presented in Table 2.

Table 1. Concrete mechanical performance.

Concrete Type	Grade	f_{cu}/MPa	f_c/MPa	f_t/MPa	E_c/MPa
ordinary concrete	C45	46.7	47	3.60	3.42×10^4
micro-expansive concrete	C45	49.6	49.6	3.81	3.44×10^4

Note: Axial compressive strength $f_c = 0.7 f_{cu}$; axial tensile strength $f_t = 0.395 f_{cu}^{0.55}$; elastic modulus $E_c = 10^5 / (2.2 + 34.7 / \sqrt{f_{cu}})$.

Table 2. Steel mechanical performance.

Steel	Diameter/Thickness (mm)	Standard Grade	Yield Strength f_y (MPa)	Ultimate Strength f_u (MPa)	Elastic Modulus E_s (MPa)
Steel bar	8	HRB400	447.9	582.6	2.04×10^5
	10	HRB400	453.3	604.6	2.12×10^5
	12	HRB400	507.2	653.3	2.00×10^5
	14	HRB400	481.8	612.7	2.05×10^5
	16	HRB400	445.5	683.2	2.10×10^5
	20	HRB400	426.2	559.1	2.13×10^5
	22	HRB400	461.8	605.0	2.03×10^5
Steel plate	8	Q345	379.5	496.1	2.06×10^5
	10	Q345	382.3	503.6	2.06×10^5
Bolt	19.85	M20 2.5 × 80	1168.20	1298.00	2.08×10^5

2.3. Test Setup and Loading Protocol

The experimental setup is shown in Figure 1. The foundation beam of the specimen was anchored to the laboratory floor using high-strength bolts. In addition, manual jacks were positioned on both sides of the foundation beam to prevent potential horizontal slip-page during testing. A column-tip loading configuration was adopted, in which a 2000 kN electrohydraulic servo actuator was connected to the column tip through threaded rods. A constant axial load was applied to the column using two hydraulic jacks. To permit lateral displacement at the column head, sliding bearings were installed between the hydraulic jacks and the reaction beam. To simulate the lateral pressure exerted by the stored grain, a horizontal hydraulic jack was used to apply an out-of-plane load to the wall panels. A distribution beam was placed between the jack and the wall surface to provide uniform load transfer and prevent localized crushing. After the vertical axial load and lateral wall pressure stabilized, quasi-static cyclic lateral loading was applied to the column top under both the force-controlled and displacement-controlled modes. The lateral grain pressure was therefore represented by the total horizontal load applied through the hydraulic jack and distribution beam. This loading method effectively accounted for the influence of out-of-plane grain pressure.



Figure 1. Experimental setup.

Before the formal test, a small-amplitude cyclic lateral load was applied twice to verify the reliability of the test setup and ensure stable data acquisition from all the sensors. Following this preloading stage, all the measurement devices were initialized. The loading protocol was established in accordance with the JGJ/T 101-2015 specification [20] (Figure 2). The vertical axial load was initially applied to 40–60% of the target value and subsequently increased to the design level of 1720 kN, corresponding to an axial compression ratio of 0.30. Five representative axial compression ratios—0.20, 0.3, 0.4, 0.5, and 0.6—were selected to cover a broad range of loading conditions from low to high axial compression levels. The composite frame specimen with prefabricated wall panels, denoted as PSRCF, was numerically evaluated under these different axial compression ratios. By comparing the responses of the same benchmark model, the effects of the axial compression ratio on the load-bearing capacity, ductility, and hysteretic behavior were assessed. The corresponding parametric results also provided insight into the collaborative load-transfer behavior of the frame and wall-panel system, although the independent contribution of the wall panels could not be isolated from the experimental sample. The load–displacement skeleton curves of the prefabricated SRC frame of the silo building under different axial compression ratios, together with the corresponding horizontal shear capacities and displacement ductility coefficients, are presented in Figure 3. The comparative results reveal a clear influence of the axial compression ratio on structural seismic performance. When the axial compression ratio was no greater than 0.4 ($n \leq 0.4$), the skeleton curves exhibited broadly similar trends, and the load-bearing capacity remained relatively stable. However, once the axial compression ratio exceeded 0.4, the descending branch of the skeleton curve changed markedly, with rapid degradation of the bearing capacity and a substantially steeper decline. In engineering applications, the design axial compression ratio of such structures should preferably be controlled below 0.4 because the cases examined here indicate that this limit is conducive to maintaining satisfactory seismic performance. Based on these considerations, an axial compression ratio of 0.3 was selected for the experiments. Simultaneously, the lateral grain pressure was applied to the wall distribution beam in 4–5 increments using a horizontal hydraulic jack until the total applied out-of-plane load reached a stable value of 200 kN. This value represented the horizontal load transmitted to the specimen's PSRCF through the hydraulic jack and distribution beam. After the vertical and lateral loading stages were completed, quasi-static cyclic loading was applied. Before the specimen reached the yield point, force-controlled loading was adopted, beginning at 10% of the estimated yield load and increasing by 50 kN at each level, with one cycle applied per increment. The yield displacement Δ_y was determined using the geometric construction method recommended by JGJ/T 101-2015. The loading mode was subsequently changed to displacement control, with multiples of Δ_y used as the displacement increments. Three complete cycles were applied at each displacement level to evaluate cyclic degradation. Loading was terminated

when the lateral load-carrying capacity decreased to 85% of the ultimate peak load or when significant structural damage occurred.

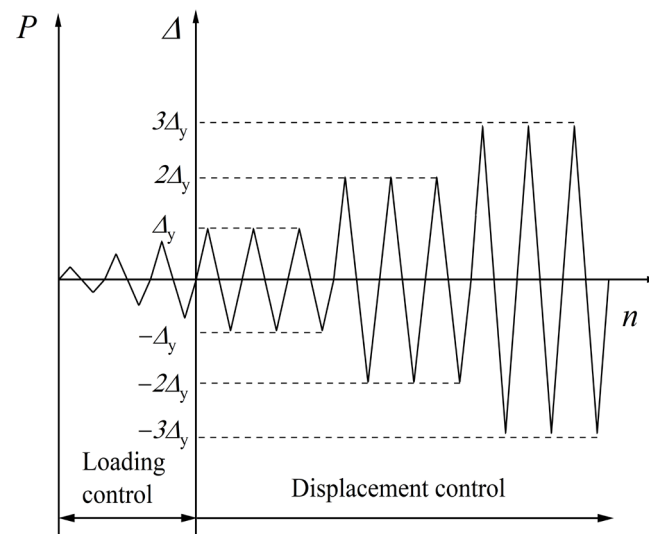


Figure 2. Schematic diagram of the loading system.

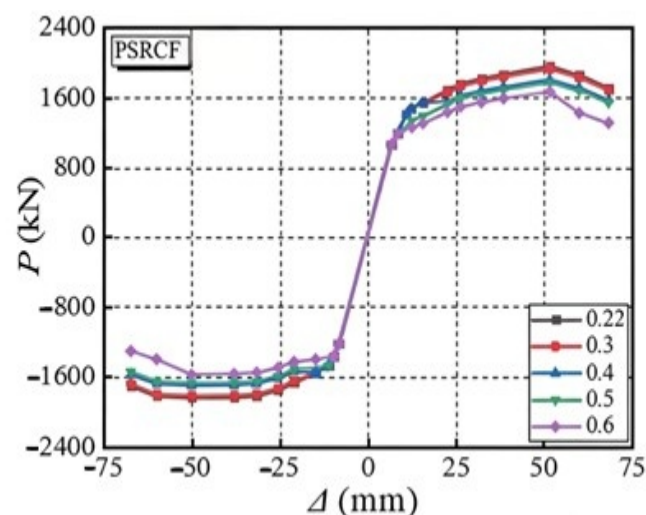


Figure 3. Effect of the axial compression ratio on the skeleton curves.

2.4. Instrumentation and Data Acquisition

The principal measurements obtained during the experiment included the horizontal load and displacement at the column tip, lateral deflections at the mid-height and base of the column, strains in the concrete and steel, and shear deformation within the beam-column joint.

The axial load applied to the column and the lateral pressure exerted on the wall panels were monitored using force sensors integrated into the hydraulic jacks to ensure that both loads remained constant throughout the test.

The horizontal load at the column tip was measured using the internal load cell of the electrohydraulic servo actuator. To capture the displacement profile, linear variable differential transformers (LVDTs) were installed at the column tip, column mid-height, and foundation beam. To assess the internal strain distribution, strain gauges were attached to the longitudinal reinforcing bars, steel sections, and concrete surfaces, as illustrated in Figure 4a–c. In addition, the shear deformation of the beam-column joint was measured using a pair of cross-arranged wire-displacement sensors (draw-wire sensors), as shown in Figure 4d.

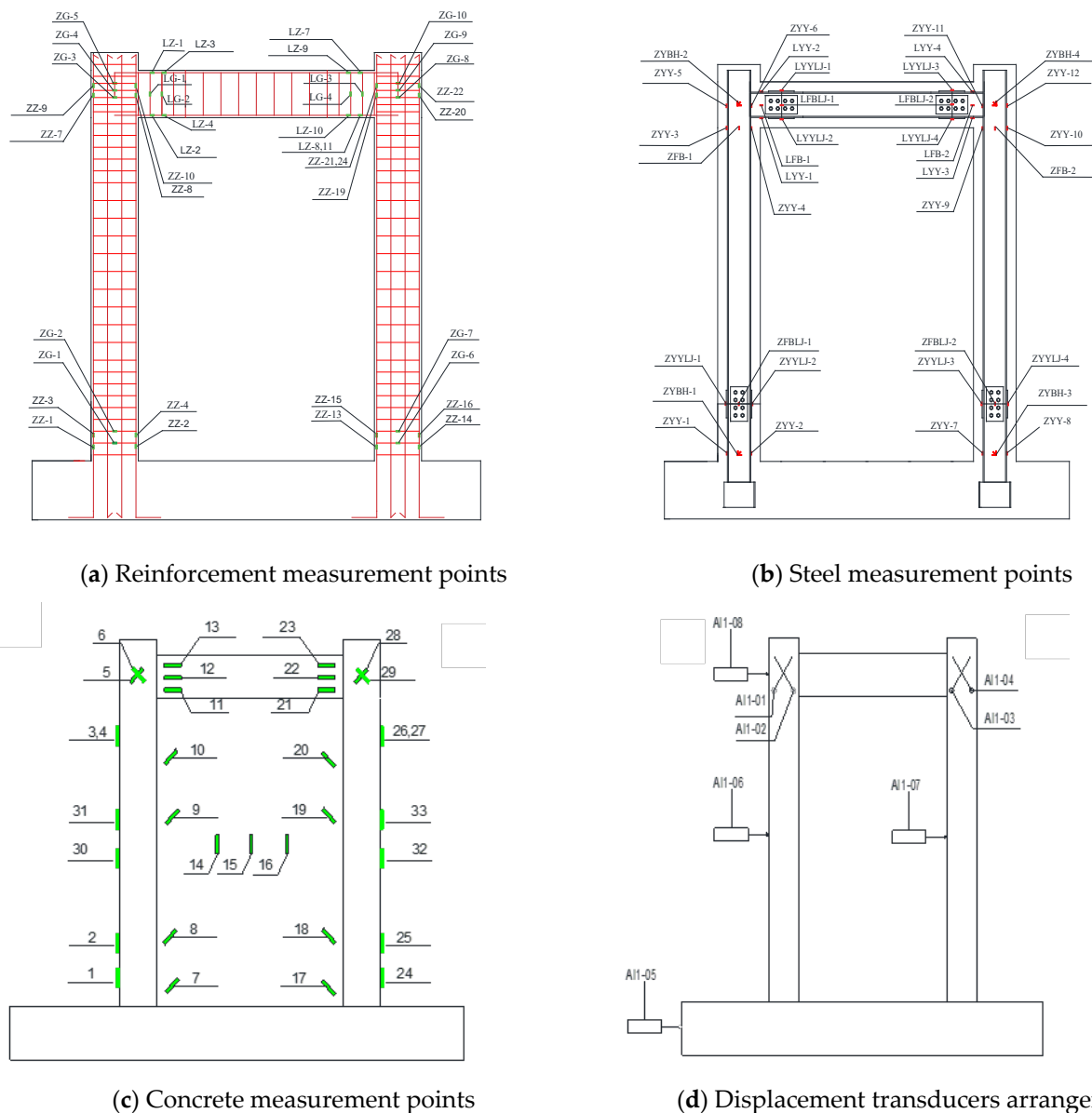


Figure 4. Specimen measurements.

3. Results and Discussion

3.1. Hysteretic Curves

The hysteretic load–displacement curves obtained under cyclic loading provide a fundamental basis for evaluating the seismic performance of structural components. These curves comprehensively reflect key mechanical indicators, including deformation capacity, energy dissipation, and strength degradation [21]. The hysteresis curves measured at the column tip of the PSRCF sample are shown in Figure 5. As shown in the figure, the measured hysteretic load–displacement curves exhibit several notable characteristics. First, the loops are generally symmetric in the positive and negative loading directions and display a full, shuttle-shaped form without obvious pinching. This behavior indicates the favorable energy dissipation capacity of the prefabricated SRC warehouse frame. In addition, the lateral resistance does not decrease abruptly after the peak load; instead, the curves exhibit a gradually decreasing branch, reflecting satisfactory ductility and deformation capacity.

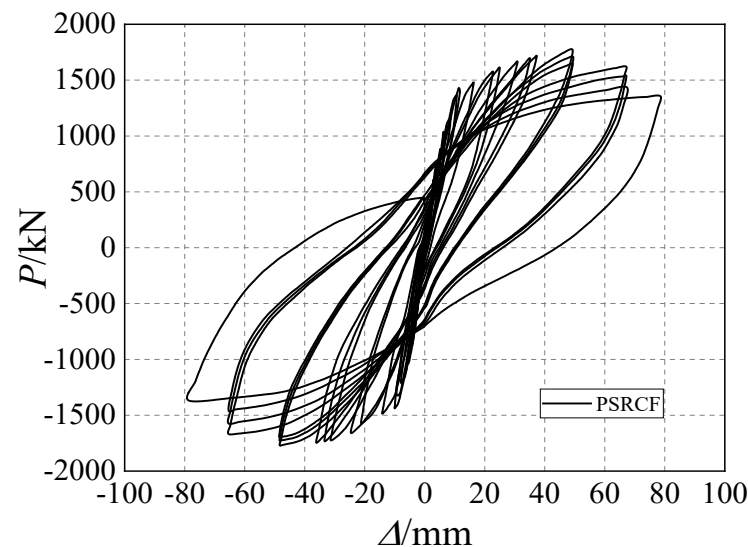


Figure 5. Load–displacement curve of specimen.

3.2. Skeleton Curves

Skeleton curves, defined as envelopes connecting the peak load points of the initial cycles at different displacement levels, directly illustrate the mechanical properties of the specimen, including load-carrying capacity, ductility, and stiffness. Consequently, they provide an important basis for evaluating structural seismic performance. Three critical characteristic points are identified on the skeleton curve: the yield, peak, and ultimate (failure) points. The ultimate lateral capacity of the PSRCF sample was evaluated from the skeleton curve obtained through quasi-static cyclic testing. The peak load (P_m) was defined as the maximum lateral resistance attained by the specimen. The ultimate load (P_u) was defined as the load corresponding to 85% of P_m on the descending branch or the load at the test termination when severe concrete crushing, spalling, or rapid strength degradation occurred. This criterion was applied in both the positive and negative loading directions, and the corresponding displacement was defined as the ultimate displacement (Δ_u). The peak point represents the maximum strength attained on the curve. In accordance with commonly adopted seismic testing standards, the ultimate point corresponds to the state at which the lateral resistance decreases to 85% of the corresponding peak value or the point at which the test is terminated because of severe damage.

The yield point was determined using the geometric plotting method (Figure 6). The yield displacement was obtained using the geometric plotting procedure recommended in the Specification for Seismic Test of Buildings (JGJ/T 101-2015). The determination procedure is summarized as follows: (1) A horizontal line AB is drawn through peak point A. (2) A tangent to the initial segment of the skeleton curve is drawn from origin O and intersects line AB at point C. (3) A vertical line is drawn downward from point C and intersects the skeleton curve at point D. (4) Line OD is connected and extended until it intersects line AB at point E. (5) A vertical line is drawn downward from point E and intersects the skeleton curve at point F. Point F is then defined as the yield point, and its coordinates represent the yield displacement and yield load, respectively.

The skeleton curve of the sample is shown in Figure 6b. Table 3 summarizes the corresponding test results. During the initial loading stage, the skeleton curves are approximately linear, with the load increasing proportionally with the displacement. Moreover, no visible cracks are observed on the specimen surface, indicating that the structure remains in the elastic stage. As loading proceeded, microcracks began to develop on the tension and compression sides of the specimen and wall panels. The rate of increase in the

load gradually becomes lower than the rate of increase in the displacement, and the skeleton curves exhibit nonlinear characteristics. Beyond the peak point, the bearing capacity decreases gradually, indicating a ductile failure mode without a sudden reduction in load. Based on the results of the geometric plotting method, the yield displacement Δ_y was determined to be 21.67 mm in the positive direction and 20.38 mm in the negative direction, as listed in Table 3. These values were consistently confirmed through repeated examination, supporting the repeatability and rationality of the determination.

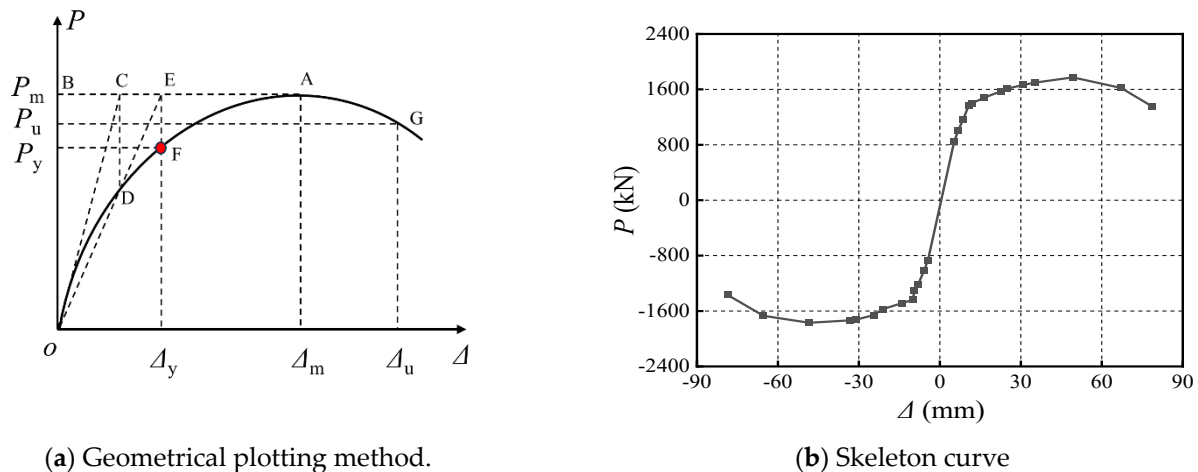


Figure 6. Experimental skeleton curve.

Table 3. Test results at each eigenvalue point.

Specimen	Yield Point		Peak Point		Failure Point	
	P_y /kN	Δ_y /mm	P_m /kN	Δ_m /mm	P_u /kN	Δ_u /mm
PSRCF	1524.69	21.67	1773.77	49.25	1507.71	68.97
	−1502.13	−20.38	−1769.38	−48.32	−1503.97	−68.59

Note: P_y , P_m , and P_u represent the yield load, peak load, and failure load of the specimen, respectively. Δ_y , Δ_m , and Δ_u represent the displacements corresponding to the yield point, peak load point, and failure point of the specimen, respectively.

To systematically quantify the degradation of the load-bearing capacity under cyclic loading, the strength degradation coefficients of the second and third cycles at each displacement level were evaluated. Taking two representative displacement levels, $3\Delta_y$ and $4.2\Delta_y$, as examples, the strength coefficients of the second and third cycles at $3\Delta_y$ are 0.963 and 0.930, respectively, whereas those at $4.2\Delta_y$ are 0.945 and 0.882, respectively. When the displacement amplitude increases from $3\Delta_y$ to $4.2\Delta_y$, the second-cycle strength coefficient decreases to 0.981 times its previous value ($0.945/0.963$), whereas the third-cycle coefficient decreases to 0.948 times its previous value ($0.882/0.930$), indicating that larger displacement amplitudes intensify strength degradation. Furthermore, at the same displacement level, the third-cycle coefficient is lower than the second-cycle coefficient by about 3.4% at $3\Delta_y$ and 6.7% at $4.2\Delta_y$. These results confirm that an increased number of loading cycles produces a non-negligible weakening effect on the bearing capacity and that this effect becomes more pronounced as the displacement amplitude increases, as shown in Figure 7.

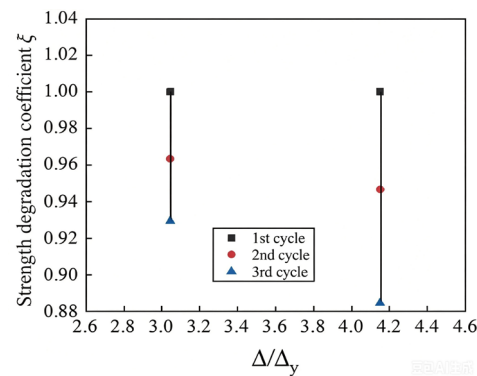


Figure 7. Strength degradation coefficient of specimens.

3.3. Energy Dissipation

Energy dissipation capacity is an important indicator for evaluating the seismic performance of structural systems because it directly influences the structural response under seismic excitation. During intense earthquakes, a structure with a strong energy dissipation capacity can effectively absorb and dissipate the input energy, thereby reducing structural damage. For the proposed prefabricated SRC grain warehouse frame, a rigorous evaluation of energy dissipation is therefore essential for assessing seismic resilience. The cumulative energy dissipation curves of the PSRCF sample, including the positive semi-cycle, negative semi-cycle, and full-cycle energy dissipation curves throughout the loading process, are shown in Figure 8.

As shown, the energy dissipation curves of the positive and negative semi-cycles exhibit a high degree of consistency. During the initial loading stage, the specimen sustained negligible damage and exhibited no visible cracking, resulting in nearly zero energy dissipation. As loading continued, the initiation of microcracks on the concrete surface caused the energy dissipation to increase gradually. After the yield point was reached, the energy dissipation capacity increased rapidly with increasing displacement level. Notably, each increase in displacement resulted in a stepwise increase in energy dissipation. At a constant displacement level, the energy dissipated during each cycle gradually decreased as the number of cycles increased, which was attributed to the accumulation of structural damage. The measured energy dissipation response of the PSRCF sample demonstrates that the integrated prefabricated SRC frame and wall-panel system has a favorable energy dissipation capacity; however, the independent contribution of the wall panels cannot be isolated from the present experimental results.

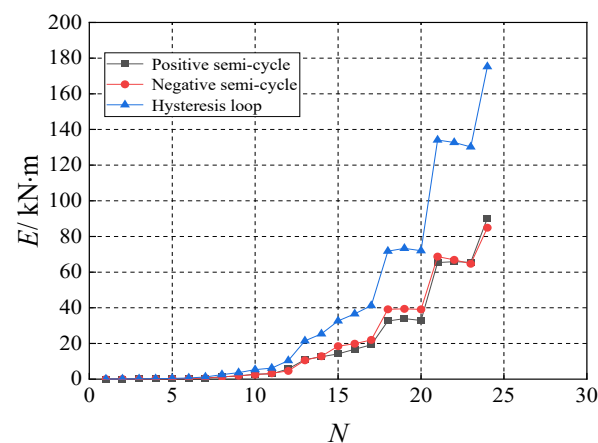


Figure 8. Energy dissipation.

In structural earthquake engineering, the equivalent viscous damping coefficient h_e is calculated according to Equation (1). In this equation, the area enclosed by the hysteresis loop represents the dissipated energy, whereas $S(OBE + ODF)$ represents the sum of the corresponding triangular areas defined by the vertices of the hysteresis loop. The calculation scheme is illustrated in Figure 9.

$$h_e = \frac{S_{ABCD}}{2\pi S_{(OBE+ODF)}} \quad (1)$$

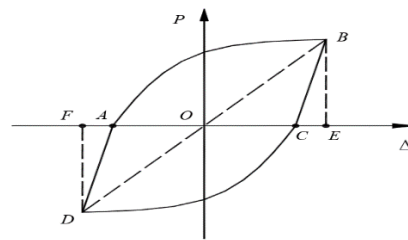


Figure 9. Calculation diagram of the equivalent viscous damping coefficient.

The equivalent viscous damping coefficient h_e was adopted as the evaluation index for the structural energy dissipation capacity. The overall value of h_e is 0.261, whereas the values at the yield point and peak load point are 0.073 and 0.132, respectively. The detailed results are presented in Table 4.

Table 4. Equivalent viscous damping coefficient of specimens.

Specimen No	h_e		
	Yield	Peak Load	Ultimate Point
PSRCF	0.073	0.132	0.261

The results indicate that the energy dissipation performance of the prefabricated SRC grain warehouse frame is favorable.

3.4. Stiffness Degradation

Lateral stiffness degradation is characterized using the secant stiffness, which is defined as the ratio of the peak load to the corresponding displacement in each loading cycle. The stiffness degradation curves are presented in Figure 10, where the ordinate represents the secant stiffness K and the abscissa denotes the normalized displacement Δ/Δ_y . As shown, the stiffness of specimen PSRCF decreases progressively with increasing displacement, exhibiting a rapid initial decline followed by a relatively stable trend during the later loading stages. Experimental uncertainties, particularly the small gap between the specimen and the actuator, affected the test results and caused a difference in the initial stiffness between the positive and negative loading directions. The initial stiffnesses in the positive and negative directions are 189.09 kN/mm and 218.22 kN/mm, respectively. This asymmetry is attributed mainly to the loading sequence: the preceding negative loading cycle induced cumulative damage and microcracks in the specimen, thereby causing pre-existing stiffness degradation during the subsequent positive loading phase.

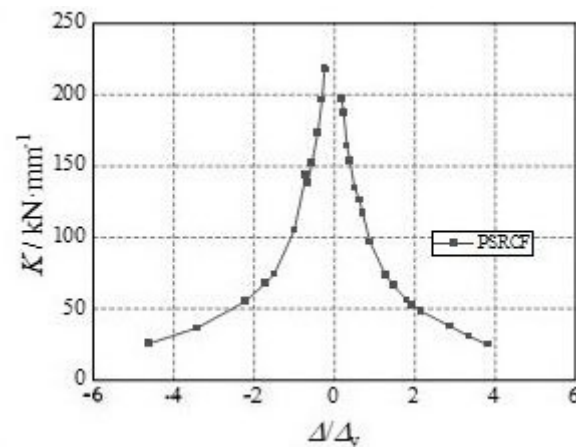


Figure 10. Stiffness degradation curve of the specimen.

4. Numerical Simulation

4.1. Constitutive Relationship of Materials

Concrete exhibits complex mechanical behavior under multiaxial stress states, and several constitutive relationships have been extensively developed to describe this behavior. The concrete damage plasticity (CDP) model [22,23] considers several critical stages, including flow, yielding, the critical state, and hardening/softening. Based on the assumption of material isotropy, the model describes the nonlinear behavior of concrete through elastic damage and tensile and compressive plasticity while accounting for stiffness degradation caused by plastic deformation during loading and stiffness recovery under cyclic load reversal.

Concrete was simulated using the concrete damage plasticity (CDP) model in Abaqus 2020. The dilation angle, eccentricity, ratio of biaxial to uniaxial compressive strength f_{b0}/f_{c0} , yield-surface shape parameter K , and viscosity parameter were set to 30° , 0.1, 1.16, 0.667, and 1×10^{-5} , respectively. The Poisson's ratio of the concrete was set to 0.2, and its elastic modulus was determined according to the calculation method recommended by ACI Committee 318-05. The compressive and tensile stress–strain relationships, together with the corresponding damage variables, were specified in the CDP model to represent stiffness degradation and nonlinear damage evolution in concrete under cyclic loading.

The constitutive relationships of the concrete adopted in the CDP model are presented in Figure 11 [24]. Before the compressive strain reaches ε_0 , the material exhibits linear elastic behavior, with an elastic limit equal to one-third of the peak compressive strength $f_c/3$. Once the strain exceeds ε_0 , the concrete enters the hardening and softening stages. During these stages, the constitutive behavior is defined by the relationship between the compressive stress and inelastic strain. Equation (2) is used to calculate the inelastic strain:

$$\varepsilon_{in} = \varepsilon_c - \sigma_c / E_0 \quad (2)$$

where σ_c and ε_c are the stress and strain, respectively, and E_0 is the initial tangent modulus (original elastic modulus).

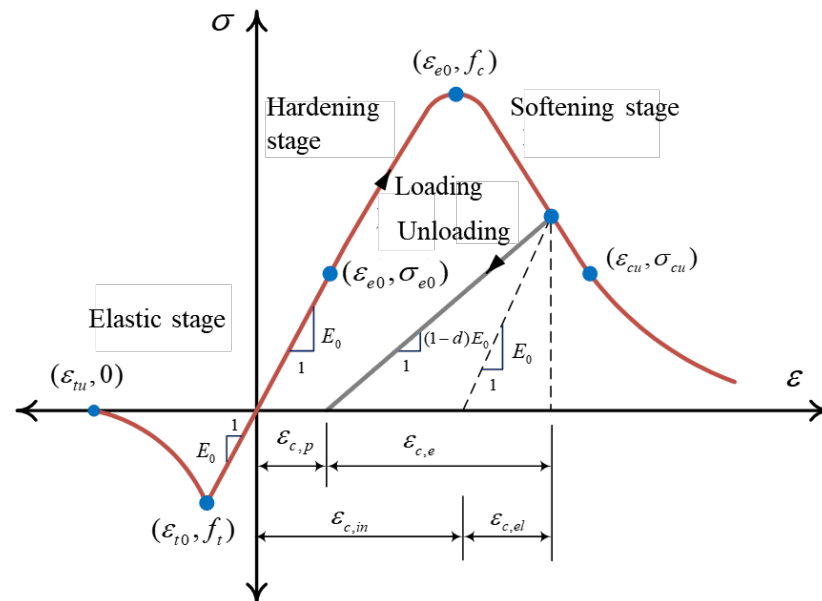


Figure 11. Constitutive relations of the concrete.

With respect to the tensile constitutive relationship of the concrete, the stress–strain response is assumed to remain linear until the peak tensile stress is reached, which represents the elastic tensile stage. The elastic modulus during this stage is assumed to be identical to the initial tangent modulus under compression. When the strain exceeds the peak tensile strain ϵ_{t0} , the concrete enters the tensile softening stage. In this stage, the stress–strain response is characterized by the relationship between the tensile stress σ_t and cracking strain ϵ_{ck} , with the cracking strain calculated according to Equation (3) [25]:

$$\epsilon_{ck} = \epsilon_t - \sigma_t / E_0 \quad (3)$$

The stress–strain relationship of the concrete under uniaxial compression specified in GB50010-2010 [26] can be determined using Equations (4)–(8).

$$\sigma = (1 - D_c) E_c \epsilon \quad (4)$$

$$D_c = \begin{cases} 1 - \frac{\rho_c n}{n - 1 + x^n} & (x \leq 1) \\ 1 - \frac{\rho_c}{\alpha_c (x - 1)^2 + x^n} & (x > 1) \end{cases} \quad (5)$$

$$\rho_c = \frac{f_c}{E_c \epsilon_{c0}} \quad (6)$$

$$n = \frac{E_c \epsilon_{c0}}{E_c \epsilon_{c0} - f_c} \quad (7)$$

$$x = \frac{\epsilon}{\epsilon_{c0}} \quad (8)$$

where α_c is the coefficient of the descending branch of the compressive stress–strain curve, f_c is the uniaxial compressive strength of the concrete, ϵ_{c0} is the peak compressive strain, E_c is the elastic modulus of the concrete, and D_c is the compressive damage parameter.

The mathematical expression for the uniaxial tensile stress–strain relationship of concrete [27] is given as follows:

$$\sigma = (1 - D_t)E_c \varepsilon \quad (9)$$

$$D_t = \begin{cases} 1 - \rho_t [1.2 - 0.2x^5] & (x \leq 1) \\ 1 - \frac{\rho_t}{\alpha_t(x-1)^{1.7} + x} & (x > 1) \end{cases} \quad (10)$$

$$\rho_t = \frac{f_t}{E_c \varepsilon_{t0}} \quad (4)$$

$$x = \frac{\varepsilon}{\varepsilon_{t0}} \quad (5)$$

where α_t is the parameter governing the descending branch of the uniaxial tensile stress–strain curve of the concrete; f_t represents the characteristic (standard) value of the uniaxial tensile strength of the concrete; ε_{t0} denotes the peak tensile strain corresponding to the tensile strength f_t ; and D_t represents the evolution parameter of uniaxial tensile damage in the concrete.

The yield function of the CDP model is expressed as follows:

$$F = \frac{1}{1 - \alpha} \left[\sqrt{3J_2} + \alpha I_1 + \beta \langle \sigma_{\max} \rangle - \gamma \langle -\sigma_{\max} \rangle \right] - \sigma_{c0} \quad (6)$$

$$\alpha = \frac{\sigma_{b0} / \sigma_{c0} - 1}{2\sigma_{b0} / \sigma_{c0} - 1} \quad (7)$$

$$\beta = \frac{\sigma_{c0}}{\sigma_{t0}} (1 - \alpha) - (1 + \alpha) \quad (8)$$

$$\gamma = \frac{3(1 - K_c)}{2K_c - 1} \quad (9)$$

where I_1 is the first invariant of the effective stress tensor, and J_2 is the second invariant of the effective deviatoric stress tensor.

The initial strength parameters σ_{c0} , σ_{t0} , and σ_{b0} represent the uniaxial compressive, uniaxial tensile, and biaxial compressive strengths of the concrete, respectively. To account for the deviatoric response, K_c is introduced to control the shape of the failure envelope on the deviatoric plane. The geometric profile of the yield surface on the deviatoric plane is defined by the parameter K , which ranges from a circle ($K = 1.0$) to a triangle ($K = 0.5$). In this study, K is taken as 0.67, which is a representative value for conventional concrete.

The CDP model adopts a non-associated flow rule, in which the plastic potential is characterized by a hyperbolic function defined as follows:

$$G = \sqrt{(\lambda \sigma_{t0} \tan \varphi)^2 + 1.5 \rho^2} + \sqrt{3} \xi \tan \varphi \quad (10)$$

where φ represents the dilation angle of the concrete in the hardening stage, which generally ranges from 33° to 42° for normal-strength concrete, and λ represents the flow-potential eccentricity, which is conventionally assigned a value of 0.1.

Concrete damage beyond the elastic limit is categorized as tensile cracking or compressive crushing. To account for the stiffness degradation associated with these damage

mechanisms, two independent damage variables, d_t and d_c , are used to scale the initial elastic stiffness. The hysteretic behavior is governed by the combined effects of the damage variables d and the stiffness recovery factors ω . Specifically, d_t and d_c represent tensile and compressive damage, respectively, whereas ω_t and ω_c characterize the degree of stiffness recovery during load reversal. The cyclic response and constitutive evolution of the CDP model are illustrated in Figure 12.

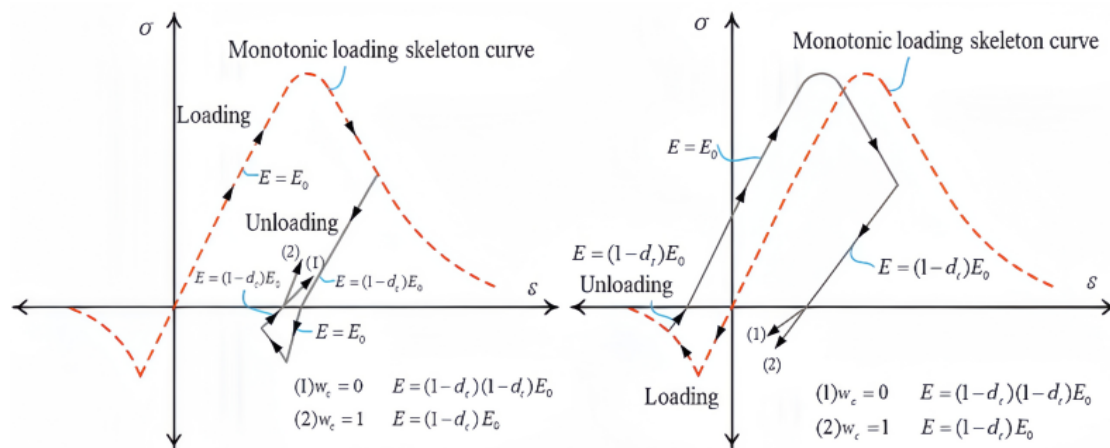


Figure 12. Hysteresis rule of concrete.

4.2. Constitutive Relationship of Steel and Bar

The principal mechanical properties of the steel were determined from material property tests. During the linear elastic stage, the stress–strain relationship exhibits a linear response. When the applied load exceeds the yield point, the steel undergoes cyclic hardening. To represent this behavior, the constitutive model incorporates the von Mises yield criterion together with a combined hardening rule. This approach combines the characteristics of isotropic hardening and nonlinear kinematic hardening [28,29]. Specifically, the isotropic component describes the expansion of the yield surface, whereas the nonlinear kinematic component represents the translation of the yield surface within the stress space, thereby effectively capturing the Bauschinger effect. A schematic representation of this combined hardening mechanism is presented in Figure 13.

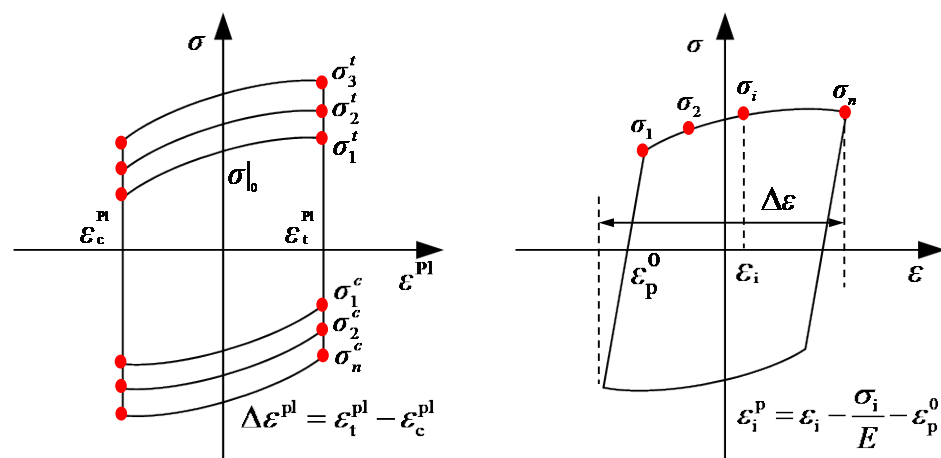


Figure 13. Calibration of material property parameters of finite element model.

Isotropic hardening describes the evolution of the yield-surface size σ_0 and is expressed as a function of the equivalent plastic strain ε_p :

$$\sigma^0 = \sigma|_0 + Q_\infty (1 - e^{-b_{iso} \epsilon_p}) \quad (11)$$

in which $\sigma|_0$ denotes the yield stress at zero equivalent plastic strain and represents the yield strength of steel; Q_∞ represents the maximum expansion of the yield surface; and b_{iso} defines the hardening rate relative to the increase in plastic strain.

4.3. Finite Element Model

In finite element modeling, the mesh density and element type directly affect the computational accuracy, convergence behavior, and solution efficiency. Considering both accuracy and efficiency, a differentiated zoned-meshing strategy was adopted for the four component types in this study. The model comprises four types of components: structural steel wall, reinforcement cage, concrete wall, and connection plates with bolted openings. The meshing rules clearly differed: a baseline mesh size of 50 mm × 50 mm was uniformly applied to the steel wall, reinforcement cage, and concrete wall, whereas only the connection plates with bolted openings were locally refined to 5 mm × 5 mm, rather than applying a uniform 5 mm mesh to the entire model. The zoned mesh arrangement is illustrated in the accompanying figure. All the mesh convergence and sensitivity analyses in this study were performed using the finite element model of the benchmark specimen PSRCF. A multidimensional mesh comparison table was constructed to simultaneously evaluate the mesh effects of the four component types, namely, steel, reinforcement, concrete, and connection plates. The overall suitability of the proposed zoned-meshing scheme was assessed from multiple perspectives, including modeling feasibility, macroscopic mechanical response, local stress accuracy of the four components, and computational cost. Detailed descriptions are provided below:

1. Design of the mesh sensitivity analysis scheme

The composite shear wall model in this study comprises four component types, each represented using a distinct element type: the concrete wall and steel skeleton were modeled using 8-node hexahedral linear reduced-integration solid elements (C3D8R); the reinforcement cage was represented using 2-node truss elements (T3D2); and the end connection plates were modeled using 4-node reduced-integration shell elements (S4R). To isolate the influence of the mesh size of each component on the structural mechanical response, a total of 12 numerical cases were established. In each case, only the mesh size of one component was varied, while the mesh sizes of the remaining components were maintained at their baseline values. For each component type, three mesh levels—coarse, baseline, and fine—were examined, with the grouping criteria defined as follows:

- (1). Concrete wall: 100 mm (coarse), 50 mm (baseline), 25 mm (fine);
- (2). Built-in steel skeleton: 100 mm (coarse), 50 mm (baseline), 25 mm (fine);
- (3). Reinforcement cage truss: 100 mm (coarse), 50 mm (baseline), 25 mm (fine);
- (4). End connection plates: 20 mm (coarse), 5 mm (baseline), and 2 mm (fine).

The baseline mesh combination corresponded to the original computational model used in this study, with a computational time of approximately 1 h 45 min per analysis. The evaluation metrics included the peak lateral load-carrying capacity, initial elastic stiffness, ultimate displacement, cumulative hysteretic energy dissipation, and computational time per model.

2. Results of the mesh sensitivity analysis for each component

- (1) End connection plates (S4R shell elements, the component most sensitive to mesh refinement)

When a 20 mm coarse mesh was used, the stress concentration and local plastic development at the plate ends could not be accurately captured, resulting in overesti-

mation of the peak load-carrying capacity and initial stiffness, with relative errors exceeding 13% for both indicators. After refinement to the 5 mm baseline mesh, the relative errors in capacity and stiffness decreased to less than 3%, and the hysteretic curves obtained from the finite element analysis substantially improved in agreement with the experimental results. Further refinement to the 2 mm fine mesh produced changes of less than 1.5% in the mechanical indices, providing only a negligible improvement in accuracy, while the computational time per model increased to nearly 4 h, thereby substantially increasing the computational cost.

(2) Concrete wall (C3D8R solid elements)

The 100 mm coarse mesh tends to smear the diagonal cracks in the wall and the stress gradients within the confined boundary zones, thereby underestimating the development of plastic damage to the concrete. The relative errors in the peak capacity and stiffness are about 5–8%. When the mesh is refined from 50 mm to 25 mm, the relative errors in all the mechanical indices decrease to less than 3%, satisfying the mesh convergence criterion. Further refinement provides only a marginal improvement in the simulation of the damage distribution, while doubling the number of elements and substantially increasing the computational time.

(3) Built-in steel skeleton (C3D8R solid elements)

The 100 mm coarse mesh cannot adequately capture local stress concentrations in the flanges and web of the steel section, resulting in an underestimation of the extent of plastic yielding in the steel. This produces positive errors of 4–7% in the overall lateral stiffness and peak capacity. With the 50 mm baseline mesh, the yielding behavior and cooperative load-transfer mechanism of the steel skeleton are adequately represented. Refinement to 25 mm results in negligible variations in the key mechanical indices and does not provide a meaningful improvement in accuracy.

(4) Reinforcement cage truss (T3D2 truss elements)

The 100 mm coarse mesh tends to weaken the simulated confinement effect of the distributed reinforcement on concrete cracking, substantially underestimating the hysteretic energy dissipation and the rate of stiffness degradation during the later loading stages. The 50 mm baseline mesh accurately captures the yielding and tensile-hardening behavior of both the longitudinal and distributed reinforcement. Further refinement to 25 mm produces fluctuations of less than 3% in the capacity and stiffness indices, with only a minor improvement in the continuity of the reinforcement stress distribution. The detailed results of the sensitivity analysis are presented in Table 5.

Finally, a uniform in-plane mesh size of 5 mm × 5 mm was adopted for the locally refined regions at the bolted beam-column joints, as illustrated in the figure below. The combination of the selected element types and mesh sizes was verified under multiple loading cases, all of which exhibited stable numerical convergence with acceptable computational cost. The concrete within the frame was modeled using 8-node hexahedral linear reduced-integration solid elements (C3D8R). The steel sections and connection plates were modeled using 4-node reduced-integration shell elements (S4R), with nine integration points through the thickness based on Simpson's integration rule. The reinforcing bars in the frame beams, columns, and foundation beams were represented using T3D2 elements (2-node three-dimensional truss elements).

Table 5. The sensitivity analysis.

Component Type	Element Type	Mesh Size	Peak Positive Lateral Load P/kN	Peak Positive Lateral Load	Peak Positive Lateral Load K/(kN/m)	Relative Error of Stiffness	Computational Time Per Model	Convergence Assessment
Concrete wall	C3D8R solid	100 mm	419.2	−5.70%	8.14	−5.70%	42 min	Non-convergent
		50 m (baseline)	444.4	−2.10%	8.63	−2.00%	1 h 45 min	Convergent
		25 mm	454	0	8.81	0	3 h 38 min	Convergent (limited accuracy gain)
Encased steel frame	C3D8R solid	100 mm	423.5	−4.80%	8.27	−6.10%	50 min	Non-convergent
		50 mm (baseline)	444.4	−1.90%	8.63	−1.70%	1 h 45 min	Convergent
		25 m	453	0	8.78	0	3 h 22 min	Convergent (limited accuracy gain)
Reinforcement cage	T3D2 truss	100 mm	427.1	−4.10%	8.35	−4.90%	45 min	Non-convergent
		50 mm (baseline)	444.4	−1.60%	8.63	−1.40%	1 h 45 min	Convergent
		25 mm	451.6	0	8.75	0	3 h 10 min	Convergent (limited accuracy gain)
End connection plate	S4R shell	20 mm	502.6	13.10%	9.62	11.50%	40 min	Non-convergent
		5 mm (baseline)	444.4	−1.50%	8.63	−1.60%	1 h 45 min	Convergent
		2 mm	437.8	0	8.49	0	3 h 52 min	Convergent (limited accuracy gain)

Notes: The relative errors of both the load-carrying capacity and initial stiffness are calculated with respect to the corresponding fine-mesh results of each component (i.e., using the fine-mesh value as the reference). The computational time per model refers to the wall-clock time for serial computing on the entire workstation. The convergence criterion is defined as follows: when the relative errors of both the peak load-carrying capacity and the initial stiffness after mesh refinement are $\leq 3\%$, the mesh independence is considered to be satisfied.

Considering that the bolted beam-column joints are the critical regions of the structure, local mesh refinement was applied to these areas, and the corresponding mesh dimensions are illustrated in the figure below. Based on empirical computational experience and iterative mesh sensitivity analyses, the mesh sizes and total element numbers for the main structural components were determined. The final mesh discretization and overall assembly of the finite element model are presented in Figure 14.

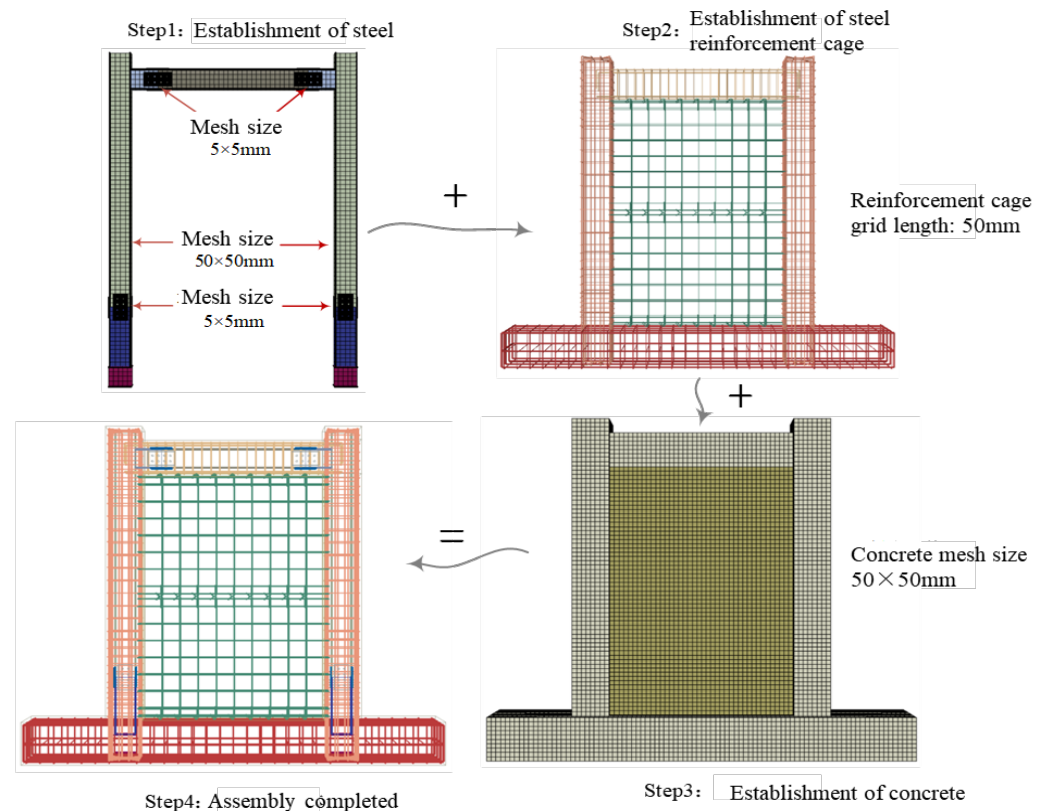


Figure 14. FE model assembly and meshing diagram.

For each component type (concrete wall, steel skeleton, reinforcement cage, and connection plate) and each mesh level (coarse, baseline, and fine), the corresponding peak load-carrying capacity, initial stiffness, relative error, and computational time are summarized in Table 5. The quantitative results indicate that compared with the other structural components, the connection plates exhibit considerably higher mesh sensitivity. When a coarse mesh of 20 mm is adopted for the connection plates, both the load-carrying capacity and stiffness are substantially overestimated, with relative errors exceeding 13%. For the concrete wall, steel skeleton, and reinforcement cage, the coarse mesh of 100 mm results in errors ranging from 4% to 8% in the principal mechanical indices. When the baseline mesh sizes are adopted for all the components, the relative errors in capacity and stiffness, compared with the fine-mesh results, remain below 3%, thereby satisfying the mesh convergence criterion. Considering both computational efficiency and numerical accuracy, the adopted mesh scheme is therefore considered appropriate for the present study.

In the FE model, Tie constraints were employed to connect the structural steel beams and columns. Embedded-region constraints were adopted between the steel sections and the concrete, assuming full composite action without considering bond-slip effects. Similarly, the reinforcing bars were embedded within the concrete host elements to ensure compatible deformation throughout the analysis. To prevent numerical penetration between interacting components during simulation, a “hard” contact property was assigned to all contact interfaces in the normal direction.

The boundary conditions and loading protocol were established to match the experimental setup. The foundation beams were fully fixed at the base. As illustrated in Figure 15, an axial load was applied to the tops of the columns, whereas lateral loads were applied at the ends of the roof and second-floor beams. These loading conditions were implemented in Abaqus through separate analysis steps. In the first step, the top surfaces of the three columns were coupled to their respective reference points (RPs), where the axial

loads were applied. In the second step, displacement-controlled lateral loading was imposed at the beam ends following the same cyclic loading history adopted in the experiment. For the parametric analyses involving out-of-plane grain pressure, an additional analysis step was introduced to apply the corresponding out-of-plane loading.

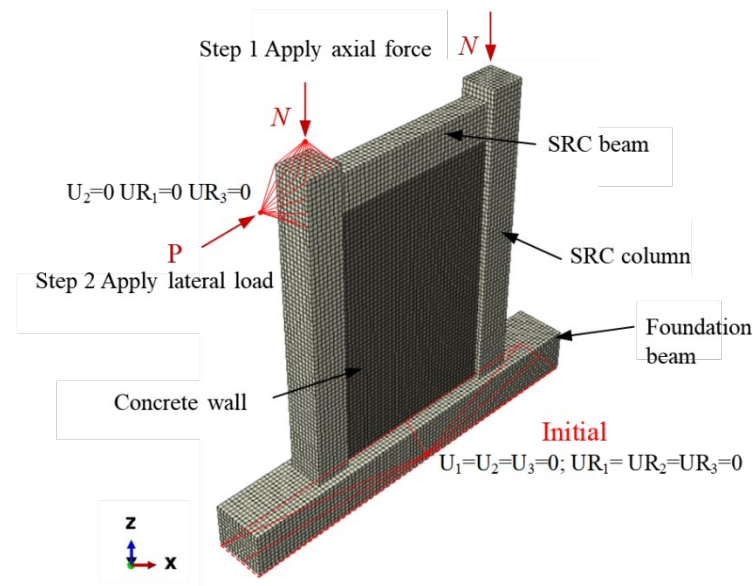


Figure 15. Boundary condition setting of the FE model.

5. Simulation Results

5.1. Stress/Strain Contour Plots

According to the low-cycle reversed loading test results of the prefabricated SRC silo frame with wall panels, the primary failure modes developed during the post-peak degradation stage. During this stage, the structural stiffness and load-carrying capacity gradually decreased as damage accumulated within the structural components. Specifically, localized crushing and spalling occurred at the base of the concrete wall panels under cyclic horizontal loading, which exhibited typical compression-crushing characteristics. Moreover, the bases of the SRC frame columns experienced concrete cracking and spalling under cyclic bending moments, representing a typical flexural failure mechanism. These observations indicate a combined failure mode characterized by crushing at the wall-panel base and flexural failure at the base of the SRC columns. The observed cracking and concrete spalling at the column base also indicate the formation of a localized plastic hinge region. Plastic deformation was primarily concentrated near the bases of the SRC columns, whereas the wall-panel bases participated in energy dissipation through localized crushing and progressive damage.

To improve the correlation between the numerical and experimental results, the global deformation at the late post-peak stage (near final failure) was selected as the principal validation benchmark. A comparison of the simulated damage contours with the observed crack patterns and failure locations provides qualitative support for the damage distribution, crack propagation, and load-transfer mechanisms predicted by the FE model. Because the strain gauge locations do not correspond directly to the nodal positions in the FE model and because the experimental strain measurements and numerical analyses were obtained at different loading stages, no point-to-point quantitative comparison between the measured and simulated strains was performed. Accordingly, the stress and strain contour plots are intended to provide qualitative verification of the overall response mechanisms rather than quantitative validation of local strain responses.

The FE analysis results are compared with the experimental observations for the PSRCF sample at the ultimate state in Figure 16. As shown, the simulated damage at the wall-panel base, characterized by material degradation and localized failure, agrees well with the observed experimental damage. Moreover, the predicted plastic hinge formation at the bases of the SRC columns corresponds closely with the experimentally observed cracking and concrete spalling. This overall agreement demonstrates that the developed FE model successfully reproduces the principal load-transfer mechanisms and failure characteristics of the structural system, supporting its applicability for subsequent parametric analyses.

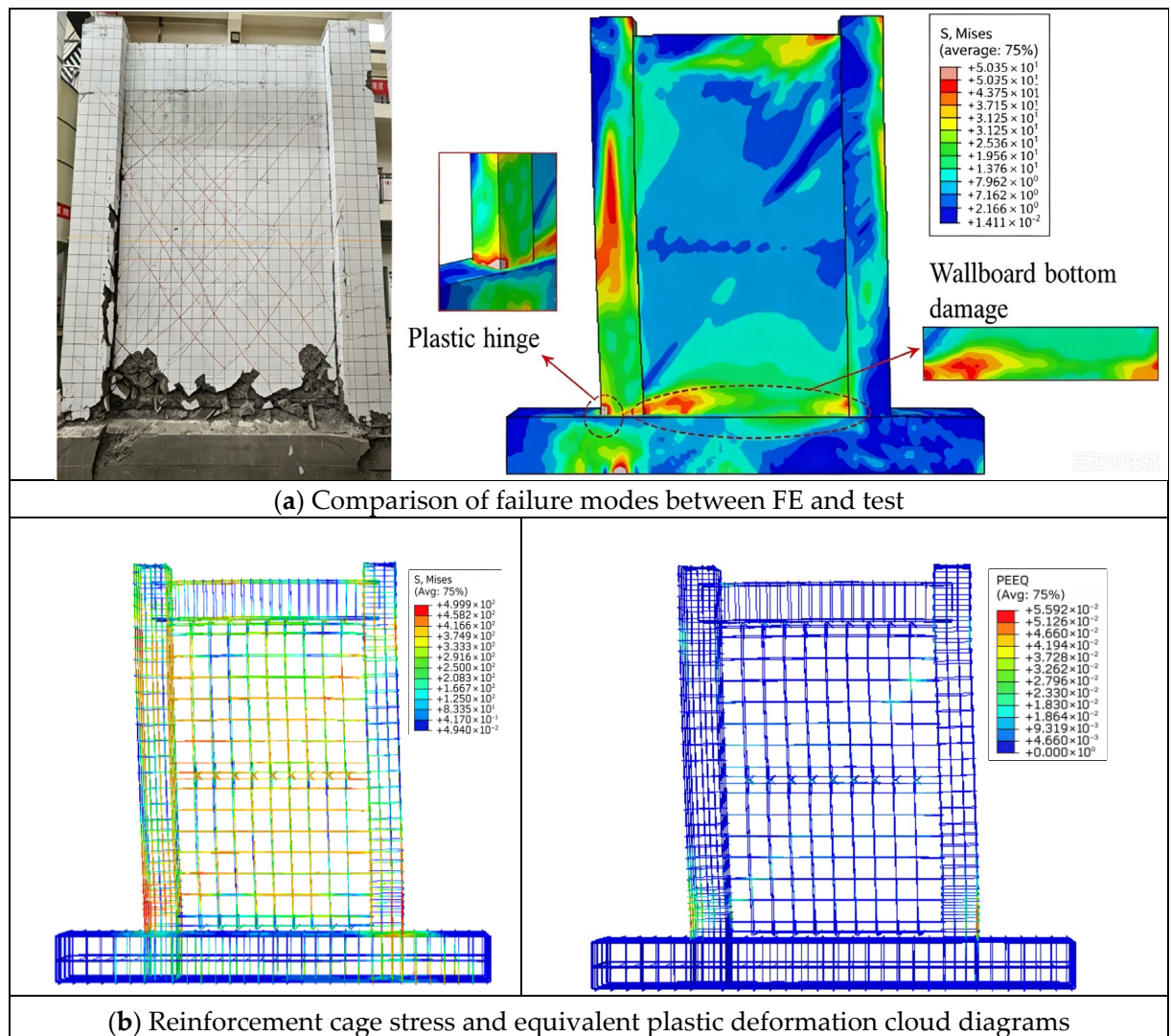


Figure 16. Failure mode and cloud diagram.

5.2. Hysteresis and Skeleton Curves

A comparison between the load–displacement hysteretic curves obtained from the FE analysis and the experimental results is presented in Figure 17a. Overall, the loading and unloading stiffness of the simulated curves are slightly greater than those of the experimental results, and this difference becomes more evident after the specimen enters the elastic-plastic stage. During this stage, the stiffness of the experimental sample rapidly decreases, whereas the FE model maintains a relatively high stiffness, resulting in noticeable differences in the hysteretic response. The experimental hysteretic loops do not exhibit a pronounced pinching effect. The FE model reproduces the overall hysteretic shape, peak resistance, and stiffness degradation trend of the tested specimen with satisfactory

accuracy. The primary discrepancy is observed along the unloading and reloading paths, which is likely associated with the simplified bond-slip and interface assumptions adopted in the numerical model. In particular, during the post-peak stage, the stiffness degradation of the experimental sample is substantially faster than that of the FE model, representing the principal difference between the two sets of hysteresis curves. These discrepancies may also be attributed to several additional factors, including the relaxation of bolt pretightening forces during cyclic loading, which reduces the global structural integrity compared with the idealized FE model, as well as unavoidable material imperfections such as local voids or honeycombing in the cast concrete that lead to lower strength and stiffness than those assumed in the simulation. In addition, the use of Tie constraints between the internal steel sections neglects the influence of welding residual stresses and local deformation associated with actual welded connections. Nevertheless, despite these differences, the FE results remain highly consistent with the experimental observations in terms of the overall deformation trend, failure mode, and load-bearing capacity, confirming that the developed model provides a reliable basis for analyzing the mechanical behavior of prefabricated SRC frame systems. The numerical and experimental skeleton curves of the prefabricated SRC silo frame are compared in Figure 17b. The numerical model predicts a yield load of 1600 kN and a peak load of 2000 kN. Although the two curves exhibit a high degree of agreement in terms of their overall development, confirming the ability of the FE model to reproduce the full-range structural response, minor discrepancies are observed during several loading stages. Specifically, the initial stiffness of the FE model is slightly greater than that measured experimentally, which is attributed mainly to the idealized constitutive models and boundary conditions adopted in the simulation. In addition, the peak load is reached slightly earlier in the numerical analysis, and the predicted peak load is somewhat higher than the corresponding experimental value. The subsequent descending branch of the numerical curve is also more gradual than that observed experimentally, indicating a slower rate of post-peak stiffness degradation. These differences are primarily associated with the idealized constitutive relationship adopted for the steel material, which may not fully reproduce the actual yield plateau and strain-hardening behavior, together with the inherent limitations of the concrete damage plasticity model in representing complex crack propagation and strength deterioration during the late stages of failure. Nevertheless, these differences remain within an acceptable range and do not affect the overall evaluation of structural performance, thereby confirming the suitability of the proposed FE model for the mechanical analysis and parametric investigation of similar prefabricated SRC structural systems.

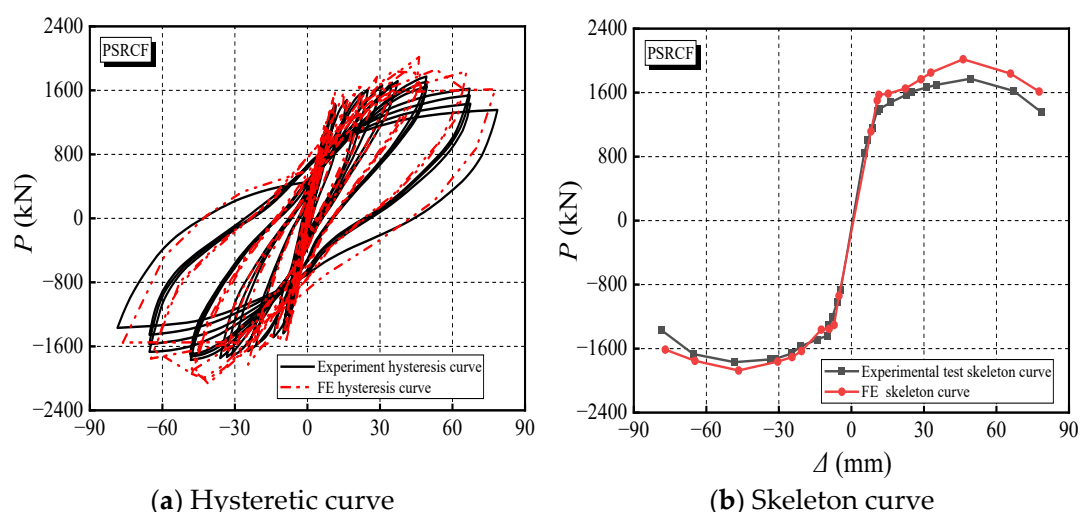


Figure 17. Comparison of the experimental results with the FE results.

6. Numerical Analysis

Given the numerous variables involved in practical engineering applications, identifying the primary factors governing structural mechanical performance is essential for providing effective guidance during structural design. For the prefabricated SRC silo frame, the mechanical behavior is influenced by multiple parameters. Therefore, this study considers two representative variables for detailed parametric analysis: the wall panel thickness and the lateral grain pressure. To rigorously evaluate the influence of each parameter, a univariate analysis strategy was adopted, in which only one target parameter was varied in each numerical case while all the other parameters were kept constant. This analytical framework enables the independent influence of each parameter on the global structural response to be clearly identified, thereby providing a reliable basis for design optimization and seismic performance assessment.

6.1. Influences of Wall Panel Thickness

In the design of prefabricated SRC silo frames, the wall panel thickness [30,31] is an important geometric parameter that significantly affects the global mechanical response and seismic performance of the structure. Appropriate selection of the panel thickness is essential for achieving a balanced distribution of structural strength and stiffness. To systematically investigate the influence of wall panel thickness on seismic behavior, a series of finite element simulations were conducted using panel thicknesses of 200, 250, 300, and 350 mm. With the specimen PSRCF as the benchmark model, a parametric analysis was performed to examine the variations in the ultimate load-carrying capacity, initial stiffness, and displacement ductility coefficient under different wall panel thicknesses. Notably, the following conclusions are derived from the finite element parametric analysis rather than direct experimental comparison.

The load–displacement skeleton curves of the prefabricated SRC silo frame under different wall-panel thicknesses are presented in Figure 18, whereas the corresponding ultimate load-carrying capacities and displacement ductility coefficients are summarized in Figure 19. Across all the numerical cases, the skeleton curves generally exhibit similar overall shapes, indicating that variations in the wall panel thickness have only a limited influence on the global failure mode and overall hysteretic characteristics. This behavior is primarily attributed to the structural hierarchy of the system, in which the wall panels function as secondary load-bearing components and generally experience localized cracking or damage before the primary frame reaches its peak resistance.

Nevertheless, the finite element results indicate that increasing the wall panel thickness leads to gradual increases in both the initial stiffness and ultimate load-carrying capacity, suggesting that thicker wall panels improve the initial lateral resistance and peak structural capacity. Moreover, the displacement ductility coefficient gradually decreases, implying that excessive wall thickness may shift the structural response toward a relatively more brittle failure mode, thereby reducing deformation capacity and energy dissipation during strong earthquakes. These numerical observations suggest that the wall panel thickness should be selected by balancing improvements in load-bearing capacity with the associated reduction in ductility to achieve satisfactory overall seismic performance.

For the PSRCF sample, the finite element parametric analysis incorporating the out-of-plane grain pressure indicates that increasing the wall panel thickness from the baseline value of 200 mm to 250 mm, 300 mm, and 350 mm produces progressive improvements in structural stiffness. Specifically, the initial stiffness increases by 20.1%, 40.5%, and 54.1%, whereas the ultimate load-carrying capacity increases by 11.2%, 19.9%, and 34.3%, respectively. Conversely, the displacement ductility coefficient decreases by 9.1%, 13.7%, and 20.2%, respectively. These numerical results indicate that under out-of-plane loading

conditions, the stiffening contribution of the wall panels becomes more pronounced, although this benefit is accompanied by a corresponding reduction in structural ductility.

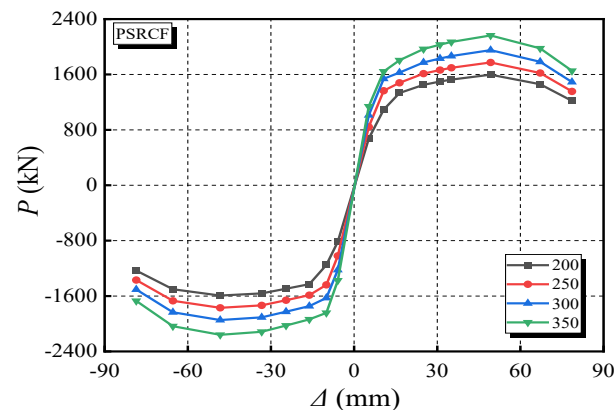


Figure 18. The impact of wall thickness on skeleton curve.

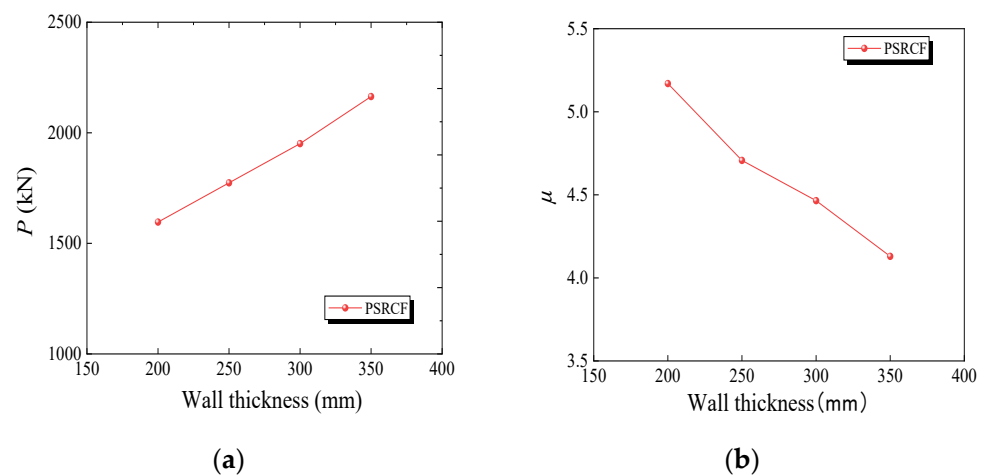


Figure 19. The influence of wall thickness on the: (a) ultimate bearing capacity; (b) node ductility.

A finite element parametric study further suggested that increasing the wall panel thickness is an effective means of enhancing the lateral stiffness and load-bearing capacity of prefabricated SRC silo frames, particularly for structures requiring high lateral stability. However, the accompanying reduction in ductility should also be carefully considered, especially under combined in-plane and out-of-plane loading conditions where the energy dissipation capacity becomes increasingly important. Therefore, the selection of wall-panel thickness in engineering design should not rely solely on strength requirements but should also consider the balance between load-bearing capacity, deformation capacity, and material economy according to the target performance objectives.

6.2. Influences of the Lateral Pressure of the Grain

This study further investigates the seismic performance of prefabricated SRC silo frame structures by considering lateral grain pressure, which represents an important loading condition in practical grain-storage facilities. Because out-of-plane grain pressure significantly influences the structural response, evaluating its effect on global mechanical behavior is essential for improving the seismic design of such structures. To quantify this influence, a series of finite element simulations was carried out using the specimen PSRCF as the benchmark model. The loading protocol incorporated four levels of out-of-plane grain pressure, namely, 100 kN, 200 kN, 300 kN, and 400 kN. The 200 kN case corresponds

to the out-of-plane grain pressure applied to specimen PSRCF during the experimental program, whereas the remaining loading levels represent different grain stacking heights or bulk densities in the numerical parametric analysis. By comparing the load–displacement skeleton curves, ultimate load-carrying capacities, initial stiffness, and displacement ductility under these loading conditions, the influence of lateral grain pressure on the principal seismic performance indices was systematically evaluated. These results provide a theoretical basis and technical reference for the seismic assessment of prefabricated SRC silo frame structures subjected to combined loading conditions.

The influence of varying lateral grain pressures on the load–displacement skeleton curves of the prefabricated SRC silo frame is illustrated in Figure 20, and the corresponding ultimate load-carrying capacities and displacement ductility coefficients are summarized in Figure 21. In the finite element model, the lateral grain pressure was applied as an equivalent strip load. The loading sequence first applied the vertical axial load together with the prescribed out-of-plane grain pressure, after which the in-plane cyclic lateral loading was imposed. The numerical results indicate that as the grain pressure increases from the baseline value of 100 kN to 200 kN, 300 kN, and 400 kN, the initial stiffness increases by 7.2%, 11.3%, and 14.7%, respectively. In contrast, the ultimate load-carrying capacity decreases by 2.8%, 9.7%, and 17.6%, whereas the displacement ductility coefficient decreases by 11.6%, 24.8%, and 39.2%, respectively. For out-of-plane grain pressures less than 200 kN, the numerical model predicts satisfactory ductility development, suggesting that this loading range is favorable for seismic performance. When the grain pressure increases to 200–300 kN, the stiffness and deformation compatibility of the structure improve, but the structure also tends to experience localized damage. At a grain pressure of 400 kN, premature yielding and a pronounced reduction in ductility are observed. These numerical results indicate that structural performance progressively deteriorates with increasing out-of-plane grain pressure.

The numerical results suggest that increasing the out-of-plane grain pressure enhances the interaction between the wall panels and the surrounding frame during the initial loading stage, causing the wall panels to participate in the resistance to lateral loads at relatively small deformations. Consequently, the overall initial stiffness of the structural system is improved. However, as the out-of-plane pressure continues to increase, the stress state within the wall panels becomes more severe, promoting localized crushing and premature damage. Once the wall panels lose part of their load-bearing capacity, the secondary lateral support provided to the frame is reduced, resulting in a decrease in the overall structural resistance. In addition, high lateral grain pressure restricts the transverse deformation and plastic development of the frame members, thereby suppressing the ductile deformation mechanism that normally develops under seismic loading. As a result, the deformation capacity and energy dissipation ability significantly decrease during the large-displacement stage, increasing the possibility of brittle failure. Overall, the results of the present experimental and numerical investigations indicate that the validated finite element model can provide a useful reference for the seismic assessment and preliminary design of prefabricated SRC grain-warehouse frames. Nevertheless, its applicability should be limited to structural configurations and loading conditions similar to those investigated in the present study.

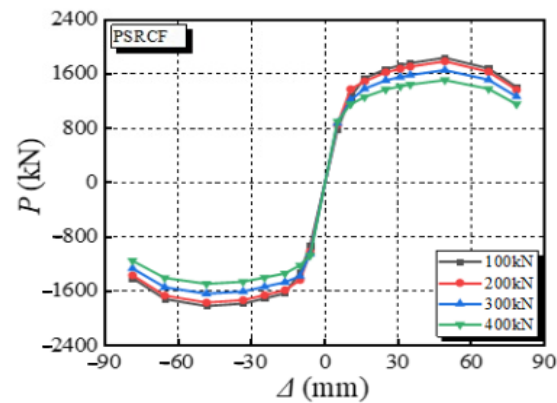


Figure 20. Effect of grain lateral pressure on skeleton curve.

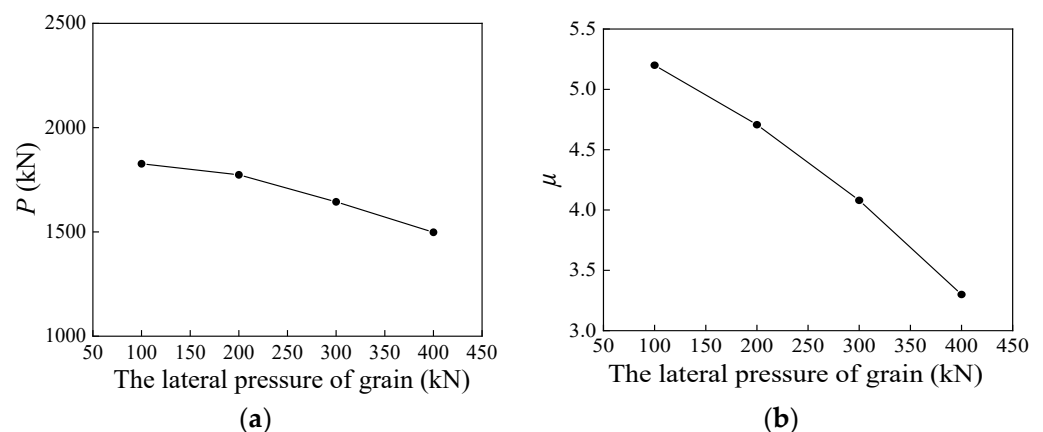


Figure 21. The influence of grain lateral pressure on the (a) ultimate bearing capacity; (b) node ductility.

7. Conclusions

In this study, the mechanical properties of prefabricated SRC multi-floored grain warehouse frames—including load-bearing capacity, energy dissipation, displacement ductility, and inter-story drift limits—were evaluated through quasi-static cyclic testing. Furthermore, finite element analysis (FEA) was performed using Abaqus to investigate the seismic damage evolution of the structural system. The principal contribution of this work is the provision of experimental evidence and a validated numerical modeling framework for the seismic analysis of prefabricated SRC grain-warehouse frames considering wall-panel interactions and lateral grain pressure.

(1) The prefabricated SRC multi-floored grain warehouse frame exhibited stable, full hysteretic loops with satisfactory symmetry in the positive and negative loading directions. No evident pinching effect was observed, indicating favorable energy dissipation capacity, with an equivalent viscous damping coefficient of 0.261 at the ultimate state.

(2) Based on the finite element parametric analysis, increasing the wall panel thickness improved the load-bearing capacity and lateral stiffness of the investigated frame system. This conclusion is derived from the numerical parametric results rather than from an independent experimental comparison of specimens with and without wall panels.

(3) The ductility of the prefabricated SRC frame was favorable, characterized by gradual post-peak strength degradation without abrupt collapse. The displacement ductility coefficients ranged from 3.15 to 3.7. The ultimate inter-story drift angles at failure (1/49 to 1/38) consistently exceeded the code-specified limit of 1/50, indicating satisfactory structural deformation capacity.

(4) The developed FE model agreed well with the experimental results in terms of the hysteretic response, skeleton curves, and failure patterns, supporting its reliability for engineering analysis. The results of the parametric analysis indicated that the wall-panel thickness substantially influences the structural stiffness, load-bearing capacity, and ductility. For the investigated specimen and loading conditions, a wall-panel thickness of 250–300 mm may provide a reasonable balance among lateral stiffness, load-bearing capacity, and deformation capacity. However, this range should be regarded only as a preliminary design reference rather than a general code-based recommendation, and further studies incorporating code requirements and economic optimization remain necessary.

(5) For the investigated specimen, the lateral grain loads adopted in the FE analysis should be regarded as equivalent out-of-plane loads under the present loading conditions. In practical design, grain pressure should be determined according to the grain type, storage height, bulk density, and applicable code provisions. The numerical results indicated that increasing the lateral grain pressure increased the initial stiffness but progressively reduced the ultimate load-carrying capacity and displacement ductility. Out-of-plane loads of 200–300 kN produced relatively favorable stiffness and deformation compatibility in the investigated model, although localized damage became more pronounced. Therefore, this range may be regarded as a preliminary upper control reference for the present structural configuration and loading conditions rather than a generally applicable design limit.

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Data Availability Statement: The experimental data supporting the findings of this study, including load–displacement curves and failure mode records, are presented in the main text of this manuscript. The raw cyclic loading data, complete material test data, strain gauge measurements, and finite element analysis input files are not publicly available due to confidentiality requirements of the research project and the large volume of data. These data are available from the corresponding author upon reasonable request.

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