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Experimental and Analytical Investigation on Seismic Behavior of Precast Modular Walls with Through-Reinforced and Bolted Vertical Connections

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Abstract: The design of inter-modular connections strongly influences the seismic performance of precast modular walls. This study investigates the cyclic behavior of three full-scale walls: S1 (integrally cast), S2 (through-reinforced grouted), and S3 (bolted), each connected to floor and ceiling slabs using vertical splicing methods. Under combined axial and lateral cyclic loading, S1 and S2 exhibited bending-dominated failures, with cracks initiating at the wall corners and propagating toward mid-height, along with superior load-bearing capacity and stiffness. In contrast, S3 experienced sliding failure at the wall-slab interface and significant interface damage. S2 outperformed S1 by 1.6% in capacity, while S3 demonstrated 36.2% lower capacity but 9% higher energy dissipation. Finite element models closely matched experimental results, with average prediction errors of 4.9% (capacity), 3.3% (displacement), and 10.6% (ductility). Parametric analysis showed that increasing reinforcement or bolt configuration significantly improved capacity and stiffness. Theoretical predictions aligned well for S1 and S2 (2.2% and 3.9% errors) but underestimated S3 due to its distinct failure mechanism. These findings provide practical insights for designing robust and resilient precast modular structures.

Keywords: Precast Volumetric Modular Structures; Inter-modular Connections; Seismic Performance of Reinforced Concrete Walls; Full-scale Experimental Testing; Finite Element Modeling; Load-carrying Capacity Models.

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1 Introduction

Traditional cast-in-situ reinforced concrete (RC) construction remains widely used but is often considered time-consuming, labor-intensive, and environmentally inefficient due to high material waste and energy consumption [1]. To address growing housing demand and sustainability challenges [2,3], precast volumetric modular structures (PVMSs) have gained prominence for offering reduced construction time, improved quality control, and resource efficiency [4,5]. PVMS involves the off-site fabrication of fully finished 3D modules, later assembled on-site, making it well-suited for mid- to high-rise applications, as shown in **Fig. 1(a-i)** [6–8]. While modular steel structures (MSS) [9–11] and their connection performance [12–17] have been extensively studied for their lightweight and efficient construction, PVMS provides better durability, fire resistance, and insulation. Recent implementations—such as the Hilton Palacio del Rio Hotel (USA) [18,19], the Nanjing Apartments and Disciplined Service Headquarters (China) [20,21], and the Clement Canopy and Avenue South Residences (Singapore)—demonstrate its potential for large-scale applications [22,23].

A critical factor influencing the seismic behavior of PVMS systems is the performance of inter-modular connections (IMCs)—mainly vertical joints between walls and floor/ceiling panels [24]. Research on traditional precast wall systems has explored various wet and dry connections, such as grouted sleeves, rebar lapping, and welded or bolted joints, with grouted sleeve connections being widely used [25,26]. Li et al. [27] demonstrated that grouted sleeve joints can offer seismic performance comparable to cast-in-place walls. Wu et al. [28] and Yuan et al. [29] found that these joints effectively transfer loads but may show reduced ductility and energy dissipation under cyclic loading. Yu et al. [30] and Gu et al. [30] revealed that such connections offer superior capacity, stiffness, and ductility compared to cast-in-place walls; however, bond-slip failure may occur under seismic excitation, especially when local bond strength is inadequate [31]. While previous

studies primarily focus on planar precast panels, PVMS systems feature integrated walls, floors, and ceilings, resulting in complex three-dimensional behavior and diverse inter- and intra-modular connections. Consequently, findings from conventional precast systems cannot be directly applied to PVMS. Further research is needed to develop advanced connection designs that ensure structural integrity, efficient load transfer, and enhanced seismic safety.

Recent studies have begun to explore the seismic behavior of modular structures with varying configurations. For example, Gunawardena et al. [32] and Shi [33] investigated MSS with and without concrete cores, emphasizing the role of stiff walls and module staggering in resisting seismic loads. Wang et al. [34–36] and Pan et al. [37,38] highlighted the importance of IMC stiffness in ensuring energy dissipation and structural stability in high-rise applications. However, much of this research focuses on numerical analyses of core-supported MSS or PVMS, often simplifying IMCs as idealized links or hinges. Experimental investigations of self-supporting PVMS—where walls, floors, and ceilings interact as a monolithic system—remain limited. Additionally, many models overlook critical factors such as realistic joint deformation, height-to-span ratio, and construction tolerances. In contrast, self-supporting PVMS equipped with robust vertical IMCs [34] show potential for improved lateral resistance and reduced reliance on time-consuming cast-in-place cores in multi-story buildings [23]. Therefore, investigating novel sustainable IMCs for self-supporting PVMS is essential for advancing modular seismic design.

Addressing this gap, the present study investigates the seismic behavior of full-scale modular walls with novel vertical splicing connections—specifically, through-reinforced grouted and bolted IMCs—to achieve efficient load transfer and enhanced seismic resilience in PVMS. Previous research has shown that through-reinforced grouted walls can achieve seismic performance comparable to cast-in-place walls by ensuring direct force transfer and reinforcement continuity

while eliminating localized bond-slip dependency [39]. This connection approach has also been implemented in the student dormitory, as depicted in **Fig. 1(a-i)**.

To evaluate the seismic performance of PVMS walls with IMCs, three full-scale specimens—an integrally cast wall (S1), a through-reinforced grouted wall (S2), and a bolted wall (S3)—were tested under combined axial and lateral cyclic loading. Based on the experimental findings, refined finite element models (FEMs) were developed, followed by a parametric study to support the development of safe and efficient design methods for multi-story PVMS applications.



Fig. 1 Stepwise construction process of prototype PVMS with proposed IMCs

2 Detailed Design of Connections and Structural Systems in Prototype PVMS

Fig. 1(a-i) illustrates the prototype project developed by the authors, forming the basis for the proposed PVMS components and connection systems. The structure comprises volumetric concrete modules, IMC technologies, and post-cast grouting. The selected prototype—the Civil

Aviation University of China Sichuan Students' Dormitory—consists of three 3-story blocks (23#, 24#, and 25#), with 234 modules (8.4×3.5×3.4 m and 8.4×3.5×3.3 m), completed in June 2023. Fully finished modules (**Fig. 1d** and **Fig. 3a**) act as standard units, with horizontal IMCs (**Fig. 3b**), vertical IMCs (**Fig. 3c**), and on-site grouting interfaces (Detail C) ensuring structural integrity and force transfer. A three-story layout was chosen due to the dormitory's temporary nature, supporting fast construction, reuse, and seismic compliance. The wall-to-wall intra-modular connection joins shorter and longer walls using through-reinforced joints. Corrugated steel pipes embedded in the longer wall receive rebars from the shorter wall (**Fig. 2**, Detail A). Before grouting, foam sealant is applied around joint edges and wall interfaces to prevent leakage and ensure pressure containment during grout injection. These joints are then filled with high-strength grout for continuity. Wall-to-floor and wall-to-ceiling connections use a bolted-grouted system (**Fig. 2**, Detail B). Each wall contains 40 mm corrugated pipes aligned with pre-formed holes in the ceiling and floor panels. Five bolts are inserted along the long edge and two along the short, totaling 28 M20 bolts per module. Grouting is performed on-site to fill annular gaps around the bolts. Foam sealant is applied before grouting to seal bolt-hole edges and pipe inlets, preventing leakage. Corridor slab support corbels are attached beneath ceiling slabs for alignment. Foam sealant is also applied at visible slab interfaces between adjacent modules to assist in positioning before grouting. Horizontal IMCs consist of U-shaped ceiling panel reinforcements lapped with longitudinal rebars and rectangular stirrups, forming a ring beam. Depending on ceiling alignment, coincident (Method 1) or non-coincident (Method 2) joint methods are used (**Fig. 3**). For vertical stacking, corrugated pipes embedded in the walls align reinforcements from base to top (**Fig. 1a**). Before grouting, foam sealant is applied at the annular gaps around pipe inlets at vertical interfaces to prevent grout leakage. After reinforcement installation, the C60 self-compacting grout is poured

into vertical and horizontal cavities to complete the load path. The quality of foam sealant application at intra- and inter-modular connections is ensured through visual inspection for complete coverage and expansion, depth gauge insertion to verify cavity fill, and spot cut-back checks to confirm adhesion and void-free filling [40]. These steps ensure effective grout containment during injection and setting.

Each 100 mm thick wall (**Fig. 3d**) integrates vertical and horizontal IMCs, intra-modular joints, and panel interfaces. Structurally, walls include two boundary columns with dense reinforcement and a coarser web mesh, serving as formwork for rebar lapping and post-casting. Gravity and lateral loads are transferred through IMCs to the ceiling panel, distributed by stiffness, and directed downward to the foundation. Precast modules (~35 tons each; **Fig. 1(a-d)**) are factory-produced, delivered by trucks (**Fig. 1e**), and lifted into position using cranes (**Fig. 1f**). Once the IMCs are aligned, they are connected, reinforcing bars are placed, foam sealant is applied where needed, and finally, the grout is poured. Floor-by-floor repetition of this process enabled all three blocks to be assembled in 19 days—achieving high efficiency via off-site finishing, pre-installed reinforcement, and elimination of on-site formwork.

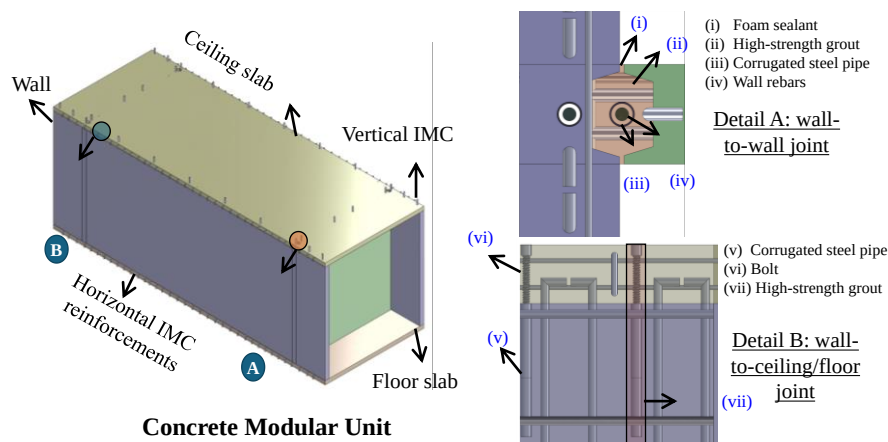


Fig. 2 Schematic diagram of wall-to-wall and wall-to-floor/ceiling slab intra-modular connections in prototype PVMS

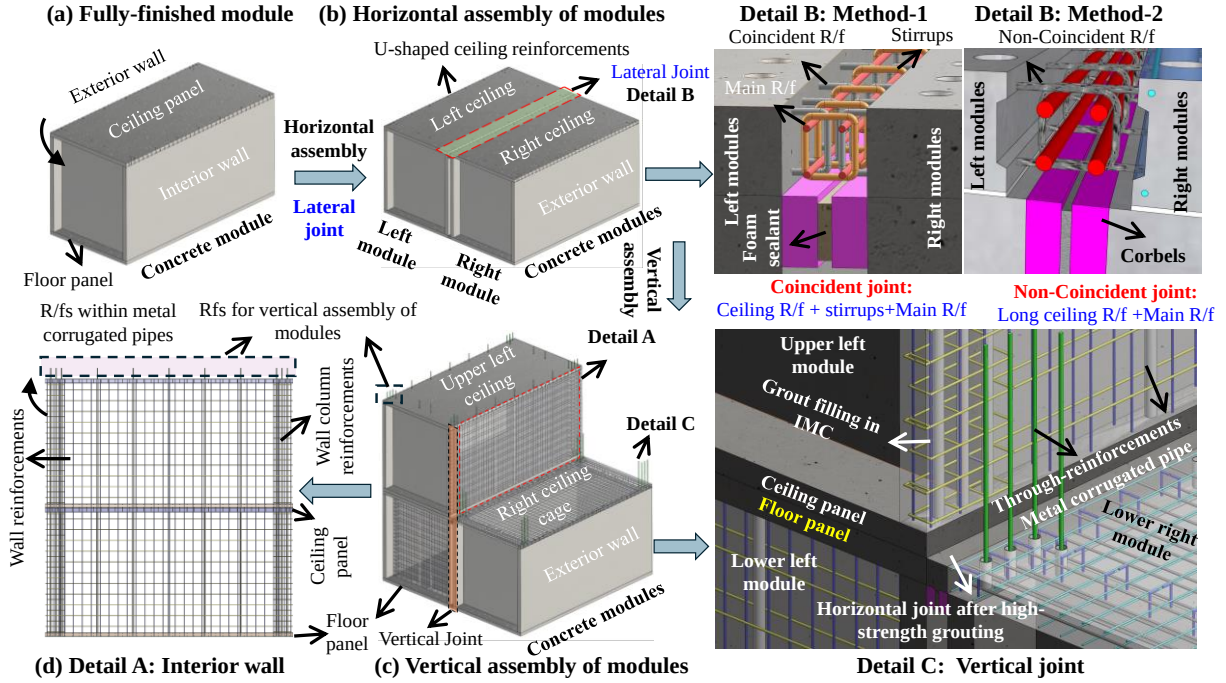


Fig. 3 Schematic diagram of horizontal and vertical IMCs in prototype PVMS

3 Experimental Study on Seismic Performance of PVMS Walls

The seismic performance of PVMS walls was evaluated through full-scale experimental testing under combined axial and lateral cyclic loading. The primary goal was to assess the hysteretic response of wall panels and IMCs, focusing on failure modes, load transfer mechanisms, ductility, and energy dissipation.

3.1 Details of Tested Wall Panels

Vertical IMCs in PVMS systems, formed by splicing prefabricated RC walls, floors, and ceilings, play a critical role in vertical load transfer and structural stability but are typically the weakest zones under seismic loads [41]. Two IMC configurations were studied: a through-reinforced grouted connection (S2) and a bolted connection (S3), both implemented at the interface between walls and floor/ceiling slabs, as shown in **Fig. 4(b, c)**. An integrally cast-in-place RC wall (S1) was also tested for benchmarking, as depicted in **Fig. 4(a)**. The through-reinforced grouted joint (S2), previously applied in real-world modular projects (**Fig. 1**), enables direct force transmission without bond slip and lap splicing, reducing material requirements while enhancing performance

in seismic zones. In contrast, the bolted connection (S3) offers easy assembly and was included for comparative purposes despite lower expected capacity. Floor and ceiling slabs were connected to wall panels to ensure structural continuity, replicate realistic PVMS diaphragm action, and evaluate vertical force transfer through IMCs.

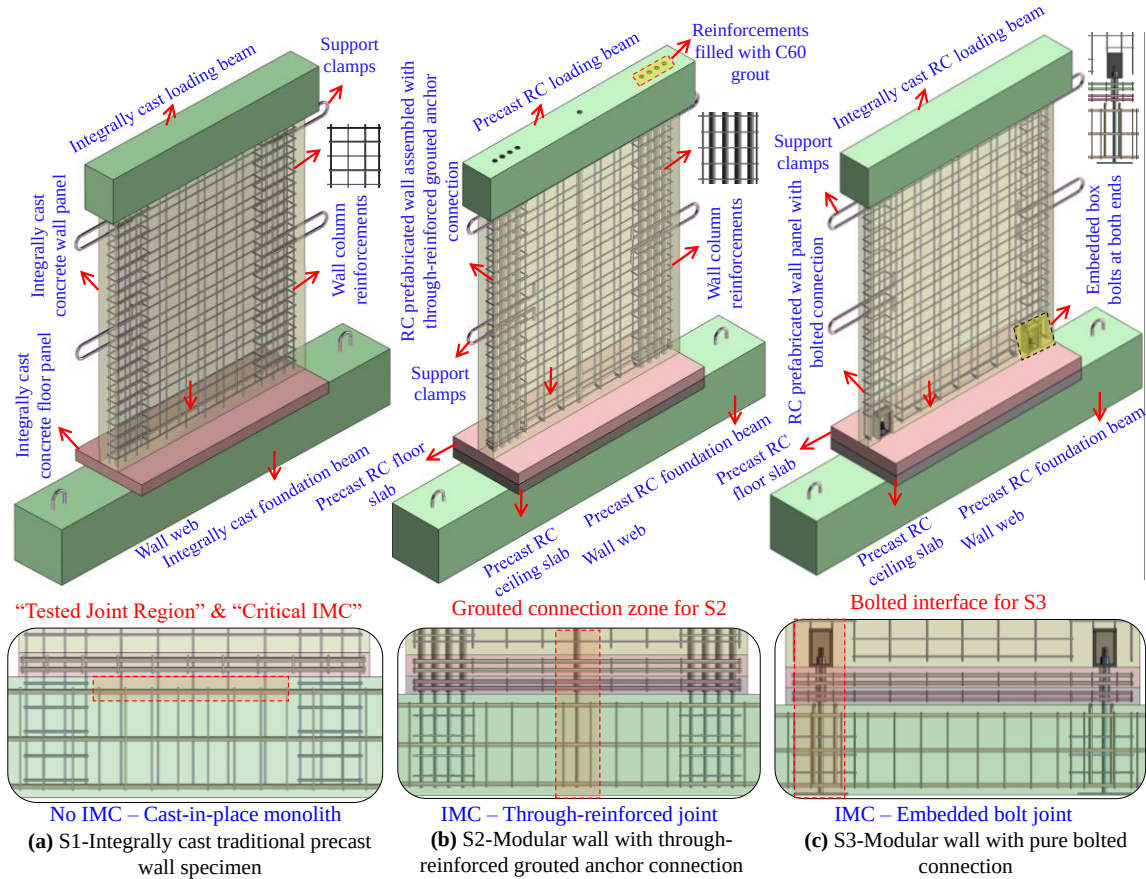


Fig. 4 Schematics of wall specimens. (a) S1: Cast-in-place wall without IMC; (b) S2: Precast wall with through-reinforced grouted IMC; (c) S3: Precast wall with embedded bolted IMC. Zoomed insets below highlight the tested joint regions

3.2 Specimen Design

Three wall specimens—S1, S2, and S3—each comprising a top loading beam, wall and slab panels, and bottom foundation beam, were designed based on an actual dormitory project. All walls measured 2300×2000×100 mm and were sandwiched between floor (ceiling) slabs sized 2000×700×100 mm (2000×700×90 mm) [42–45]. The loading beam (2200×300×400 mm) and foundation beam (3510×500×600 mm) remained constant. The total height of S1 (S2 and S3) was

3300 mm (3400 mm). Floor and ceiling slabs extended 300 mm, loading beams 100 mm, and foundation beams 755 mm for anchorage and integrity. The wall column (web) rebar region was 350 mm (1200 mm). The loading beams had $\text{Ø}20 \times 4$ mm longitudinal reinforcements ($\text{Ø}8 \times 24$ mm stirrups), measuring 2150 mm in length (350×250 mm cross-section). The foundation beams used $\text{Ø}20 \times 10$ mm longitudinal reinforcements ($\text{Ø}8 \times 56$ mm stirrups) with a length of 3460 mm (420×280 mm cross-section). S2 employed $\text{Ø}40 \times 9$ mm, 2.7 m long corrugated pipes to guide $\text{Ø}18 \times 9$ mm and $\text{Ø}6 \times 1$ mm through-rebars (3375 mm) along edges and midsection. S3 used two 40 mm diameter metal pipes with M30 bolts (30 mm dia, 850 mm length, Q355 steel), designed under EN 1993-1-8 [46], AISC 360-16 [47], and GB 50017 [48] standards. The floor and ceiling slabs had 1950 mm long $\text{Ø}6 \times 8$ mm longitudinal bars with $\text{Ø}6 @ 200$ mm stirrups. Stirrup dimensions were 650×70 mm (floor) and 650×55 mm (ceiling), with bar sizes matching the wall's transverse reinforcement. Wall webs used $\text{Ø}6$ mm; edge columns used $\text{Ø}8 \times 16$ mm bars with $\text{Ø}6 \times 68$ mm stirrups at 100 mm spacing and 375×64 mm transverse ties. Central wall longitudinal bars were 2650 mm long. All rebars had standard bend lengths of 3.5Ø . **Fig. 5(a–c)** shows reinforcement and geometric layouts, with parameters summarized in **Table 1**.

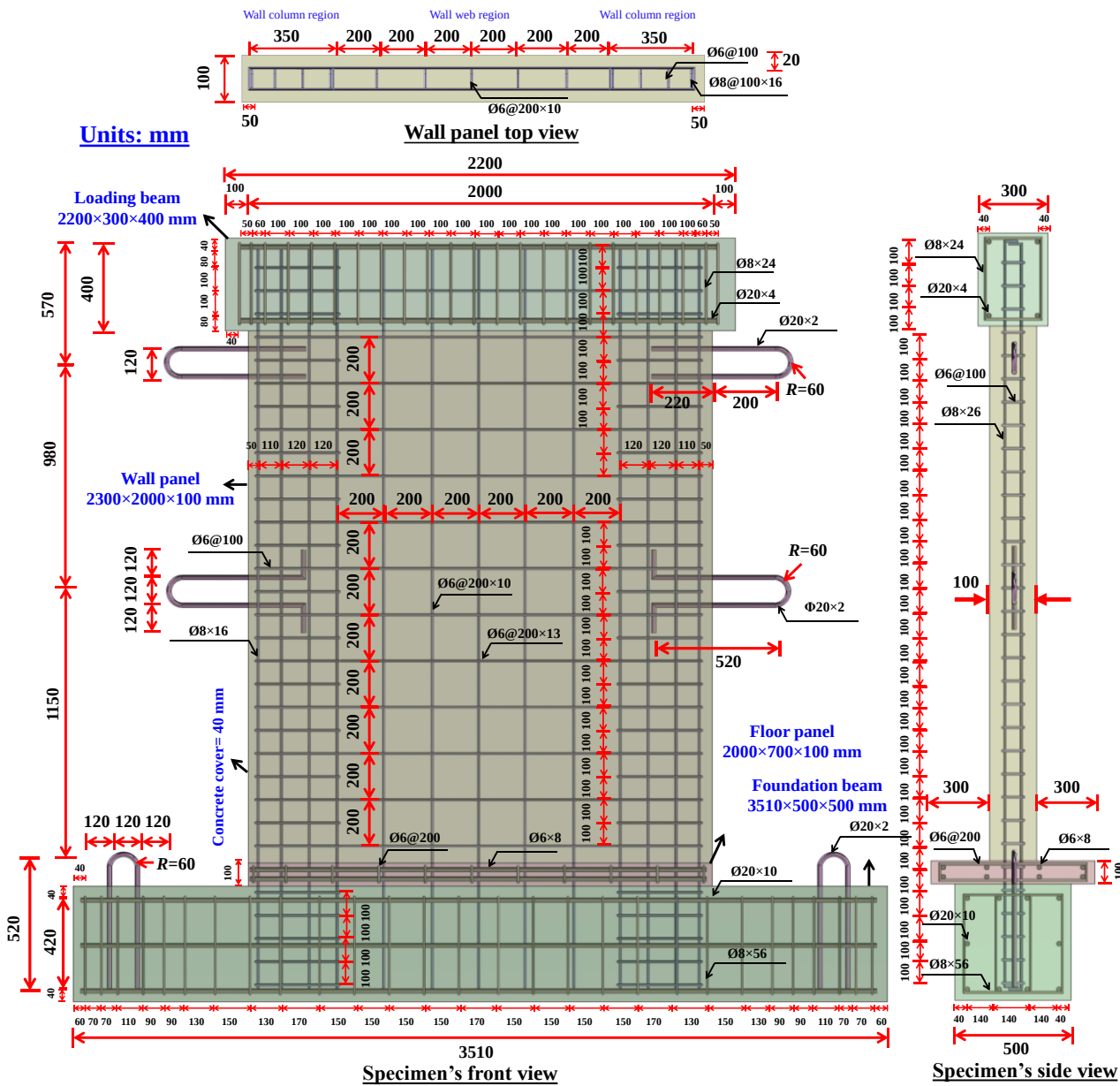
Table 1 Detailed design parameters of S1, S2, and S3 wall-tested specimens

Item	Wall size (mm)			Aspect ratio	Width ratio	Compression ratio	Wall Rebars (mm)	Concrete strength	Rebar strength	Loading method
	L_w	B_w	t_w							
S1	2300	2000	100	1.15	20	0.3	$\text{Ø}8 @ 100$	C40	HRB400	Cyclic
S2	2300	2000	100	1.15	20	0.3	$\text{Ø}8 @ 100$	C40	HRB400	Cyclic
S3	2300	2000	100	1.15	20	0.3	$\text{Ø}8 @ 100$	C40	HRB400	Cyclic

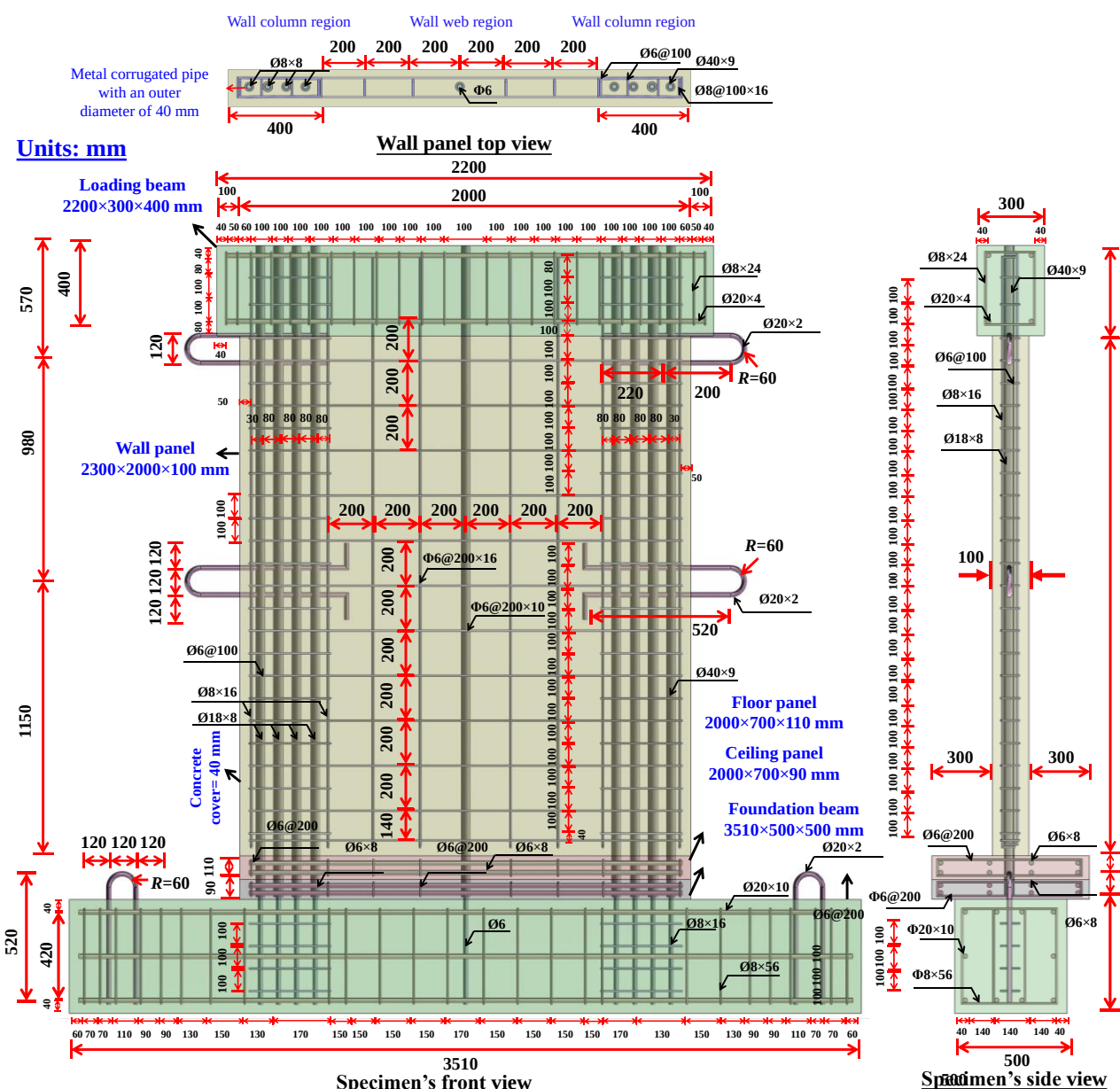
Note: L_w , B_w , and t_w represent the height, width, and thickness of the tested specimens' wall panels.

Table 2 Mechanical properties of concrete used in tested specimens

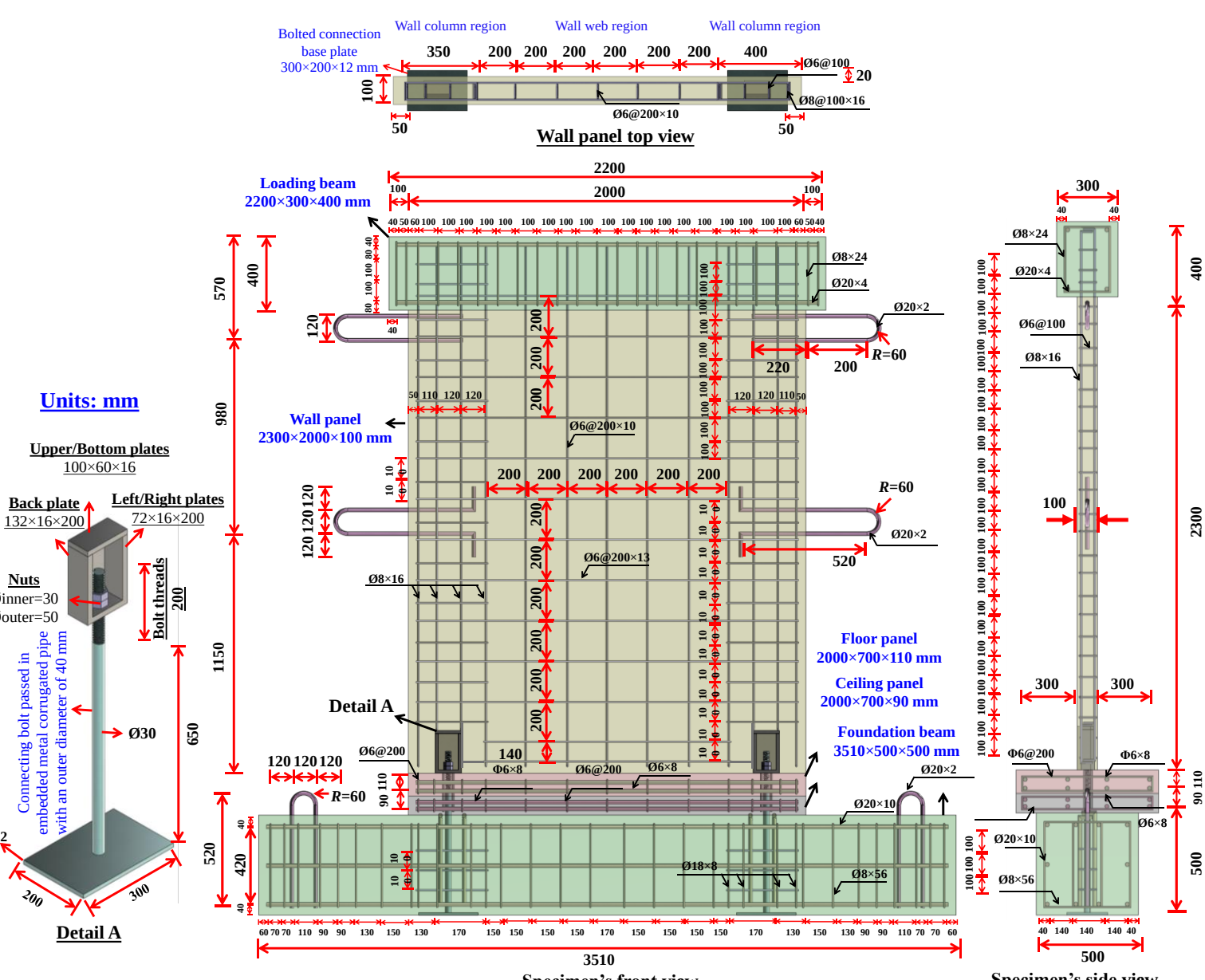
Item	$F_{u,c}$ (kN)	$f_{cu,m}$ (MPa)	f_c (MPa)	f_t (MPa)	Note: $F_{u,c}$, $f_{cu,m}$, f_c , f_t , $F_{u,f}$, and f_{cf} represent ultimate compressive load, average compressive strength, axial compressive strength, axial tensile strength, ultimate flexure load, and axial flexural strength, respectively.
Precast concrete	986.25	43.8	29.3	9.2	
Post-cast concrete	991.50	44.1	29.5	9.2	
Grouting material	1616.25	71.8	47.8	15.3	
Item	$F_{u,f}$ (kN)	f_{cf} (MPa)			
Grouting material	2808.80	10.53			



(a) Design dimensions and reinforcement details of S1



(b) Design dimensions and reinforcement details of S2



(c) Design dimensions and reinforcement details of S3

Fig. 5 Front, side, and top views of the S1, S2, and S3 wall-tested specimens, showing embedded concrete strain gauges, LVDT arrangement, and structural drawings (Scale: 1:1 and units: mm)

3.3 Material Testing

All concrete panels were cast using C40-grade concrete, while C60 grout was used for IMCs. Material properties were tested per GB/T 50081-2019 [49], with three 150×150×150 mm concrete cubes and three 40×40×160 mm grout specimens cast alongside the panels and cured for 28 days. Compressive strength (f_{cu}) and grout flexural strength (f_{cf}) were obtained and used for FEM input. All reinforcement bars were HRB400 grade steel. Tensile tests were conducted on 450 mm long samples per GB/T 228.1-2021 [50] with strain and displacement gauges installed for elongation measurements. Average stress values were calculated from the actuator load and bar cross-sectional area. Mechanical properties for concrete, grout, and steel are presented in **Tables 2–3**, with test setups and failure modes shown in **Fig. 6(a–c)**. The axial compressive strength of concrete (f_c) and axial tensile strength (f_t) were computed based on GB 50010-2010 [45] using **Eqn. 1**:

$$f_c = 0.88 \times 0.76 \times f_{cu,m} ; f_t = 0.88 \times 0.395 \times f_{cu,m}^{0.55} \quad (1)$$

where f_c , $f_{cu,m}$, and f_t represents the axial compressive strength, average compression strength, and axial tensile strength of concrete obtained from compression tests.

Table 3 Mechanical properties of reinforcements and stirrups in tested specimens

Item	f_y (kN)	f_u (MPa)	E_s (GPa)	Note: f_y , f_u , and E_s represent the yield strength, ultimate strength, and elastic modulus of steel reinforcements and stirrups used in S1, S2, and S3 tested specimens.
Ø6	453.4	655.1	257	
Ø8	504.8	708.3	250	
Ø12	447.4	613.9	242	
Ø18	486.3	682.6	235	
Ø20	448.4	631.8	226	

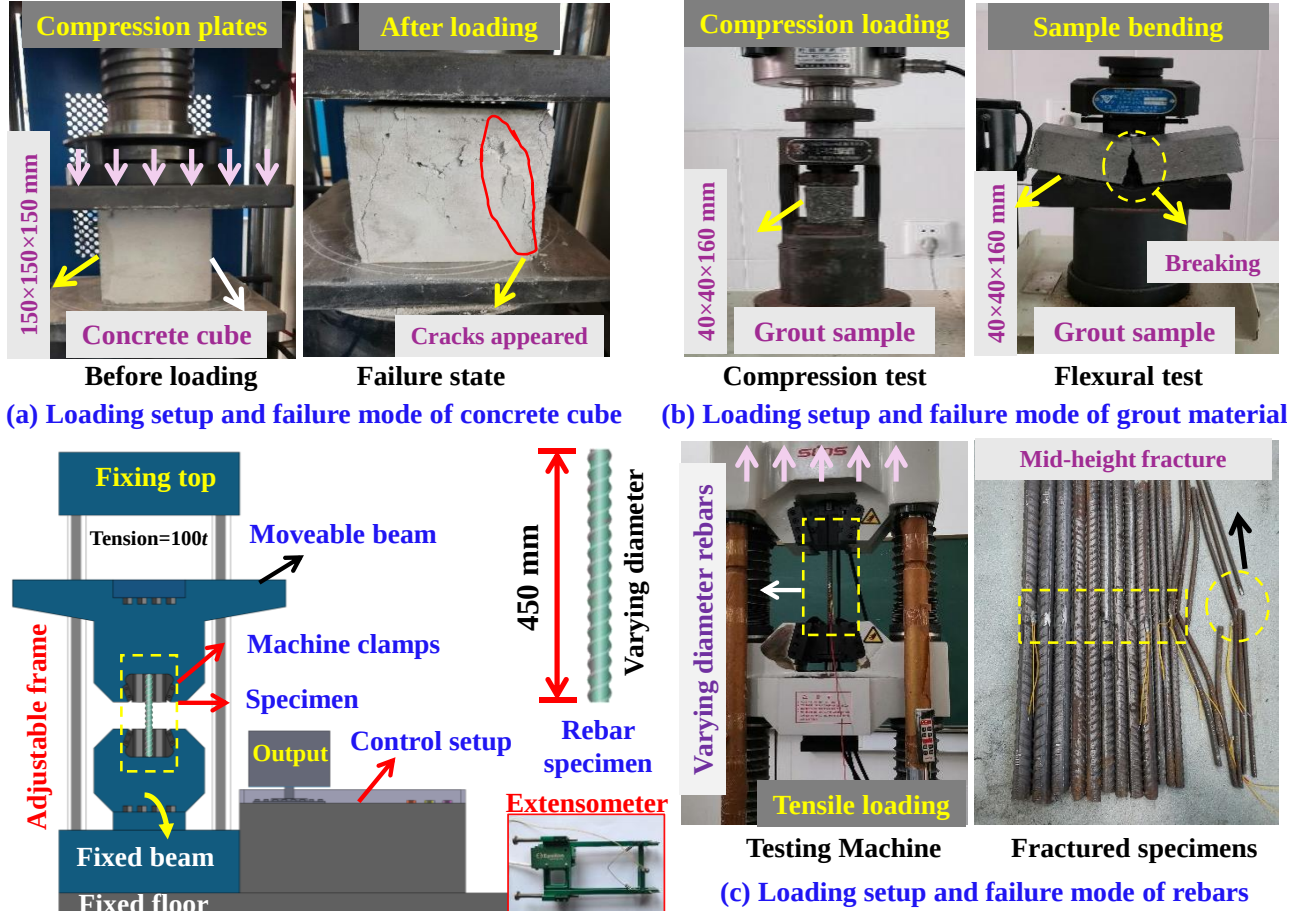


Fig. 6 Material testing setup and failure modes for concrete, grout material, and rebar specimens

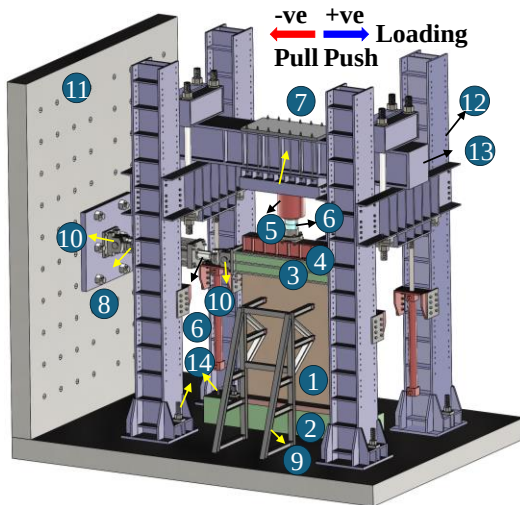
3.4 Testing Setup

Cyclic loading was conducted in the structural engineering lab using the test setup in **Fig. 7(a,b)**. A constant axial load with a compression ratio of 0.3 (**Table 1**) was exerted via a vertical hydraulic jack connected to a loading beam and roller support, allowing lateral movement during horizontal loading [51]. Axial load was applied in three steps—33%, 66%, and 100% of the target load—and held constant during lateral loading (**Fig. 8a**). Lateral cyclic loads were applied using a bi-directional horizontal actuator hinged at both ends, with the foundation beam anchored using eight bolts. White paint and a 50 mm grid were applied to the wall surfaces for crack observation, and a side-mounted lateral restraint frame with low-friction rollers minimized out-of-plane displacement (**Fig. 7e**). The axial compression ratio, n_d , is calculated per Ref. [42] using the formula $n_d =$

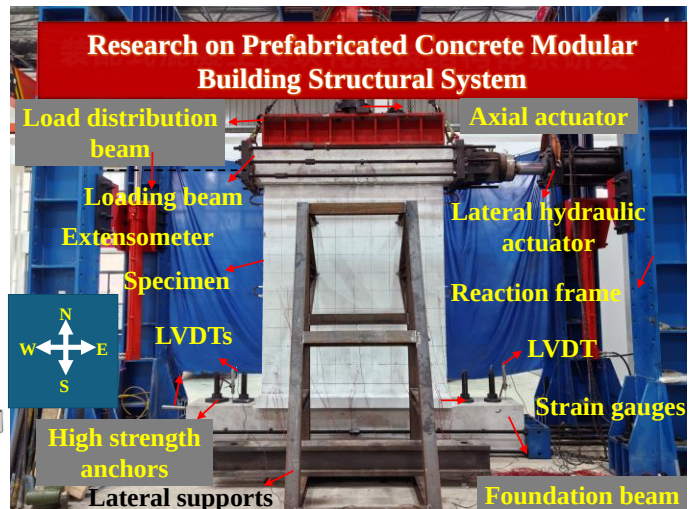
204 $\frac{N_d}{f_c A_c + f_y A_s}$, where N_d is the axial load, f_c and f_y are compressive and yield strengths, and A_c and A_s
 205 are concrete and steel areas.

206 3.5 Loading Protocol

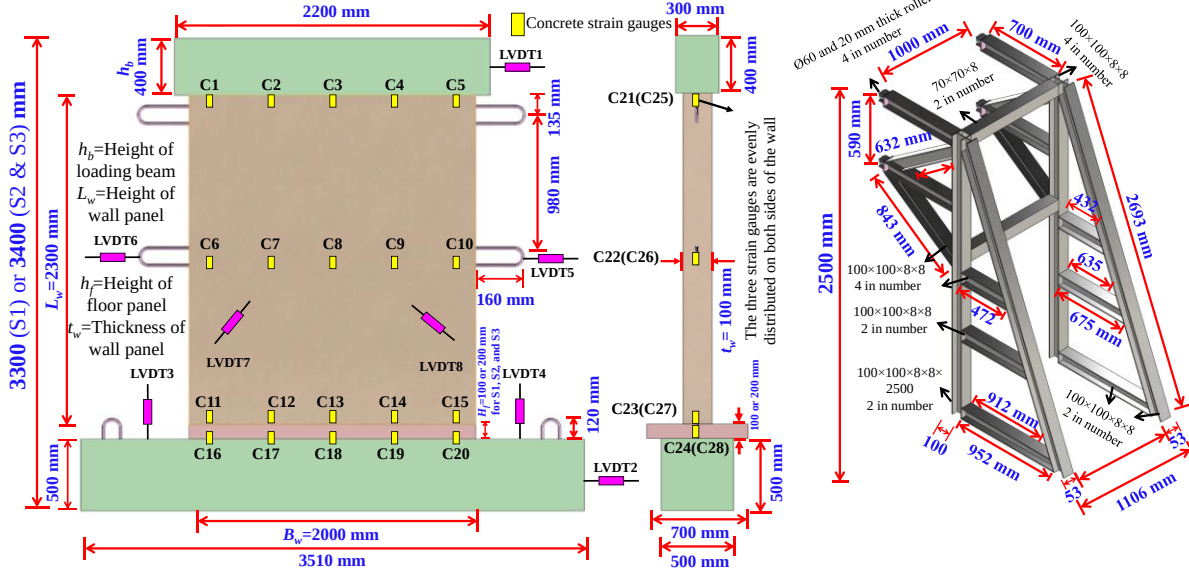
207 The loading procedure followed GB/T 50344-2019 [52] and JGJ 101-2015 [53], comprising
 208 preloading and formal testing phases. Preloading verified equipment functionality using three
 209 force-controlled cycles of 50 kN ($\leq 10\%$ of ultimate load, P_u) applied at 50 kN/min. Each increment
 210 was held for 2 minutes and unloaded gradually. After preloading, a constant axial load—applied
 211 in three stages via vertical actuators—was maintained throughout testing. Lateral cyclic loading
 212 was then applied using a horizontal actuator. Before yielding, five force-controlled cycles were
 213 applied: $1/5P_y$, $2/5P_y$, $3/5P_y$, $4/5P_y$, and P_y . After yielding (when bottom rebars on both sides began
 214 yielding), the protocol switched to displacement-controlled loading with three repetitions per drift
 215 level. The yield drift ratio, θ_y , was defined as the lateral displacement (Δ) divided by the effective
 216 wall height (H). Testing continued until the specimen's load capacity fell below 85% of the peak
 217 or severe damage occurred. The loading protocol used in this study is illustrated in Fig. 8(a).



(a) Test setup schematic diagram



(b) On-site view of test setup and specimen

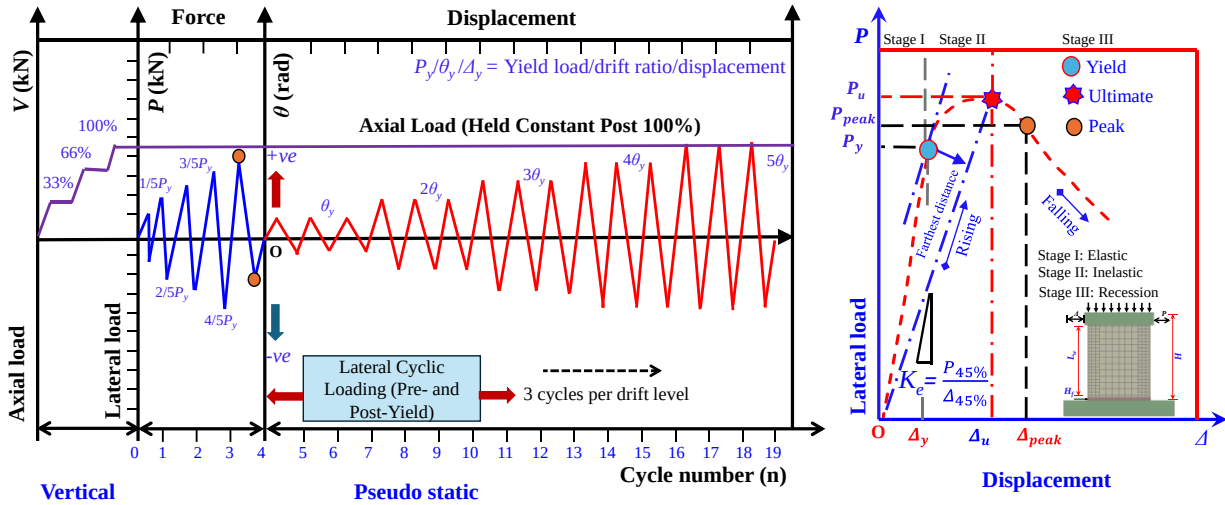


(c) Specimen front view (d) Side view (e) 3D view of lateral support

Fig. 7 General seismic loading test setup for RC wall specimens. **Note:** 1-Wall specimen; 2-Foundation beam; 3-Lateral loading beam; 4-Axial load distribution beam; 5-Axial load actuator; 6-Load sensor; 7-Roller support; 8-Lateral hydraulic actuator; 9-Lateral supports with roller wheels; 10-Hinged joint; 11-Load reaction wall; 12- Reaction frame; 13-Reaction beam; 14-High strength anchors.

3.6 Measuring Instruments

Measurements included axial/lateral loads, displacements, and strains. Load sensors mounted on the actuators captured applied forces. Strain gauges (120 Ω) were affixed to reinforcement bars and concrete surfaces using 502 adhesives and sealed with silicone rubber and epoxy-impregnated gauze for protection during casting. Grid sizes were 3 $\times$ 2 mm (steel) and 3 $\times$ 3 mm (concrete). Gauge placements are shown in **Figs. 5(a–c)** and **7(c,d)**. Displacement was monitored using eight LVDTs and extensometers (see **Fig. 5(a–c)** and **Fig. 7(b,c)**). LVDT1 measured lateral displacement at the loading point, while LVDTs 5 and 6 tracked global wall deformation. LVDT2 captured foundation slippage, and LVDTs 3 and 4 monitored rotation and overturning. Diagonal shear deformation was measured using LVDTs 7 and 8 across the two wall sections.



(a) Loading protocol for RC wall-tested specimens (b) General yield moment method
Fig. 8 Cyclic loading protocol and yield moment method for RC wall specimens

3.7 Experimental Outcomes

3.7.1 Failure Patterns

The observed failure mechanisms and responses of the wall specimens, as shown in **Fig. 8(b)**, can be categorized into three distinct phases: linear elastic (Stage 1), nonlinear (Stage 2), and degradation (Stage 3). Initially, in Stage 1, when axial compression or force-controlled lateral loading was applied, no noticeable deformation or cracking occurred, and the specimens remained in a purely elastic state. As the load increased through Stage 2 and Stage 3, yielding initiated, and cracks developed. These cracks were carefully recorded in terms of width and number on both the front and back faces of the wall. For consistency, the north (front) and south (back) faces were designated according to the laboratory orientation, with the east (loading side) and west (opposite) used throughout. Push and pull directions correspond to positive and negative displacements, respectively.

3.7.1.1 Specimen S1

No significant cracks or deformation were observed during the preload and axial compression phases. Upon lateral loading, the first two horizontal and diagonal cracks appeared at 1.24 mm

displacement on the north face. As displacement reached -2.49 mm, more cracks developed at mid-height. A diagonal crack emerged on the south face at 2.74 mm, followed by a horizontal crack at -4.46 mm. Diagonal and horizontal cracks were visible at 6.66 mm at the east corner, widening to 0.14 mm, and later similarly at the west corner by -9.13 mm. At 10.12 mm, multiple cracks with 0.2 mm width developed. At -12.12 mm ($\theta = 1/214$), yielding of longitudinal reinforcement was recorded, transitioning the specimen into displacement-controlled loading. In the $1 \times \Delta_y$ cycle, a through-diagonal crack (0.58 mm width) formed. At $1.5 \times \Delta_y$, concrete crushing and reinforcement buckling were initiated at the lower right corner. This intensified with cyclic loading until capacity fell below 85% of peak, prompting test termination. The final failure mode is shown in **Fig. 9**.

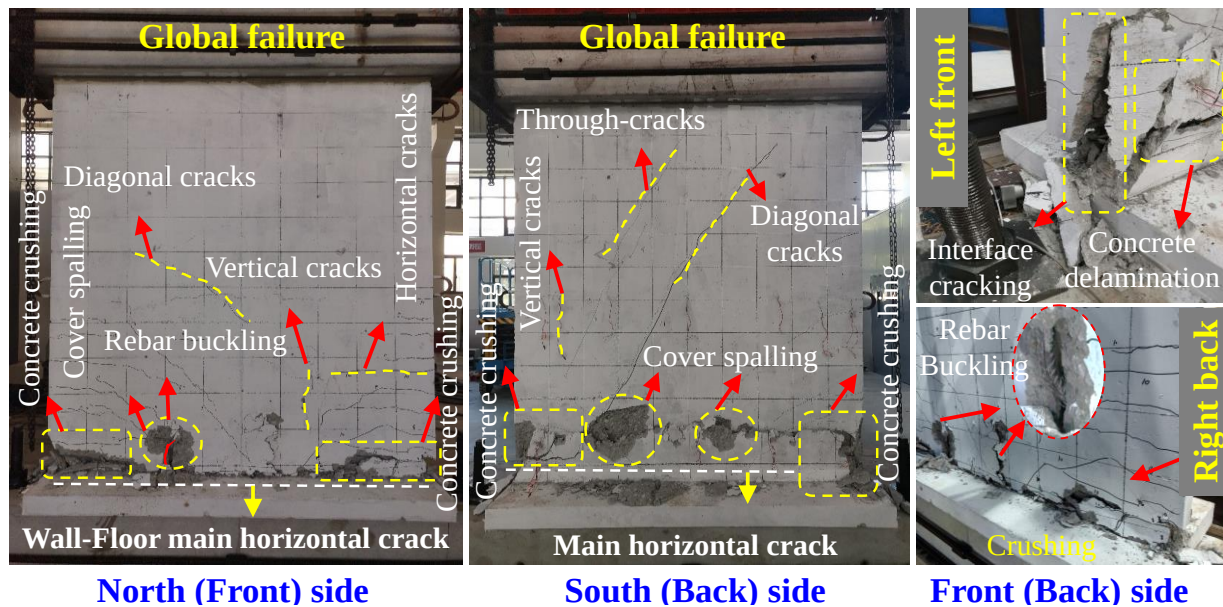


Fig. 9 Failure modes and crack patterns of S1

3.7.1.2 Specimen S2

Specimen S2 showed elastic behavior through preloading and axial compression. Vertical and diagonal cracks initiated at the east root at 10.64 mm lateral displacement ($\theta = 1/244$). Reinforcement yield followed, prompting a transition to displacement-controlled loading. At $1 \times \Delta_y$, a diagonal crack extended from the corner. At $1.5 \times \Delta_y$, additional diagonal cracks formed at the

corners, and new horizontal and diagonal cracks appeared centrally. Audible cracking accompanied by widening to 0.2 mm. During $2\times\Delta_y$, more vertical and diagonal cracks emerged, and existing ones widened substantially. With further cycling, capacity dropped below 85% of the ultimate, and the test was stopped. The final failure modes of S2 are shown in **Fig. 10**.

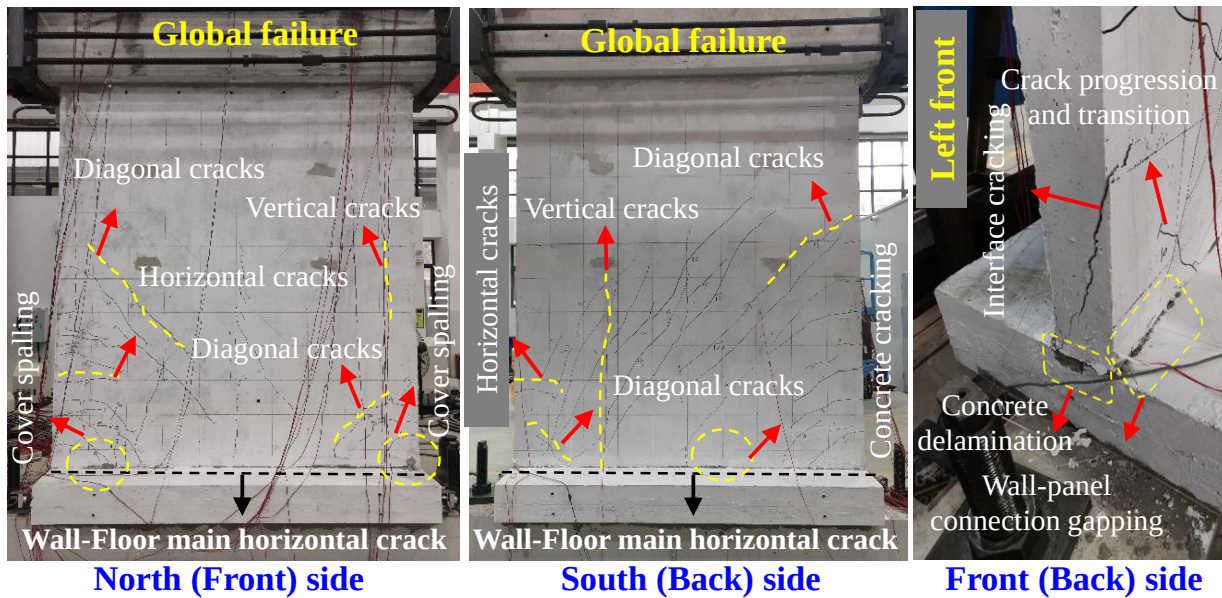


Fig. 10 Failure modes and crack patterns of S2

3.7.1.3 Specimen S3

S3 also remained crack-free during preloading and axial compression. At a displacement of -2.64 mm, a diagonal crack formed on the west slab. By -5.27 mm, diagonal cracking appeared on the east slab, accompanied by vertical cracks along the slab edges and initial concrete spalling. At 6.14 mm, a new diagonal crack developed on the east slab. The wall's longitudinal reinforcement yielded at -9.75 mm ($\theta = 1/267$), transitioning the loading protocol into displacement control. During $1\times\Delta_y$, new diagonal cracks formed along the slab edges. At $1.5\times\Delta_y$, slight spalling was observed on the west side. By $2.5\times\Delta_y$, multiple diagonal and vertical cracks emerged, and sliding between the wall and floor slab reached 2.08 mm. With continued displacement ($3\times\Delta_y$), sliding increased to 5.04 mm, alongside crack propagation and concrete bulging on the east slab. At $3.5\times\Delta_y$, the displacement between the corrugated pipe and slab reached 8 mm. By $4\times\Delta_y$, wall-slab sliding

reached 8.75 mm, with increased cracking and base spalling. At $4.5 \times \Delta_y$, sliding extended to 11.34 mm (west) and 7.37 mm (east), while the embedded pipe slipped 5 mm. This interface slip was primarily attributed to the absence of roughened contact surfaces. Still, the bolted connection maintained load path integrity due to bolt pretension and frictional engagement, operating within its deformation limits. The current S3 configuration supports rapid assembly in low-risk, short-service applications (e.g., temporary housing with ~5-year design life), where modules may be disassembled or replaced rather than repaired post-earthquake. While S3 maintained its connection integrity without catastrophic failure, the observed sliding emphasizes the need for interface enhancements—such as roughening, shear keys, interlocks, or energy-dissipating devices—for higher seismic regions. Conversely, S2 demonstrated superior seismic performance, substantial force transfer, and minimal deformation, making it suitable for long-term service in high-seismic zones (≥ 50 years), as used in our pilot project.

Conclusively, all three specimens (S1, S2, and S3) exhibited bending-dominated failure modes. Cracking was typically initiated at corners and propagated diagonally under increasing drift. Horizontal and diagonal cracks, concrete crushing, and yielding of longitudinal reinforcement confirmed flexural failure dominance. S3 additionally showed sliding failure at the floor-wall interface. These outcomes underscore the importance of robust reinforcement detailing and IMC configuration to control bending, shear, and interface deformation, particularly under cyclic seismic actions.

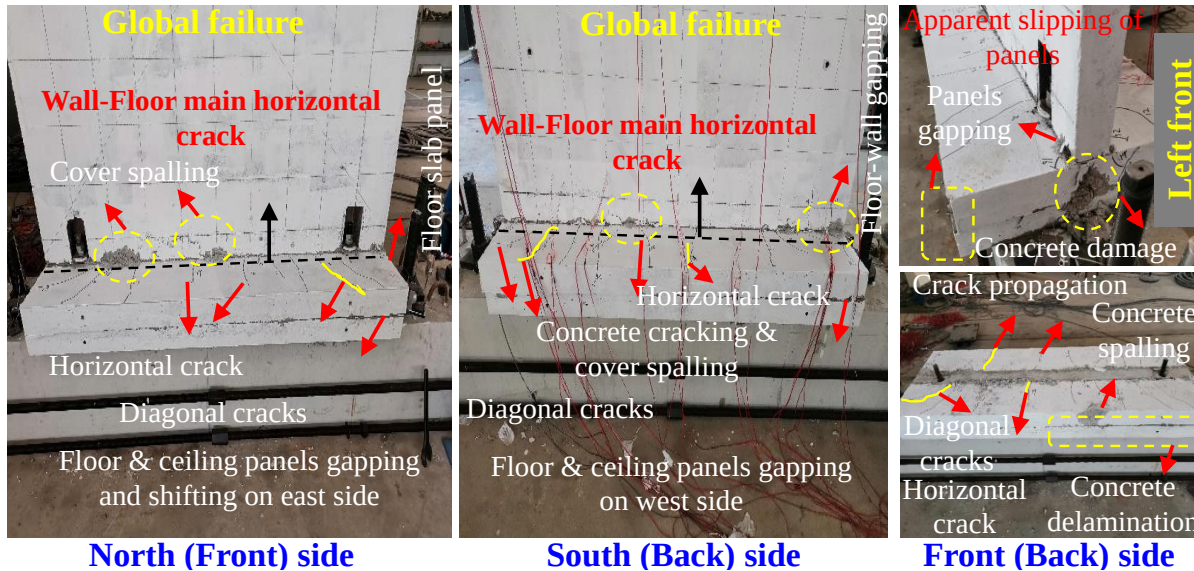


Fig. 11 Failure modes and crack patterns of S3

3.7.2 Load-Displacement Responses

3.7.2.1 Hysteretic Curves

Hysteresis curves are essential indicators of seismic performance, capturing deformation characteristics, stiffness degradation, energy dissipation, and pinching effects under cyclic loading. **Fig. 12(a–d)** displays the hysteretic responses of S1, S2, and S3. All curves demonstrated bow-shaped loops with varying degrees of pinching, primarily due to bond slip between concrete and rebars or interface slip in connected panels. In the initial elastic phase (Stage 1), the curves remained linear with small loop areas, indicating limited residual deformation. As cracking initiated and increased (Stage 2), the loops expanded, showing growing energy dissipation and stiffness reduction. Significant concrete spalling and bar exposure occurred in the failure stage (Stage 3), particularly in S3, resulting in widened loops and severe degradation. S1 and S2 showed controlled plasticity concentrated in the wall panels, whereas in S3, damage concentrated near the floor slab interface, consistent with cracking and sliding patterns shown in **Figs. 9–11**. The pinching effect was most severe in S3 due to interface slip and reduced stiffness at bolted joints.

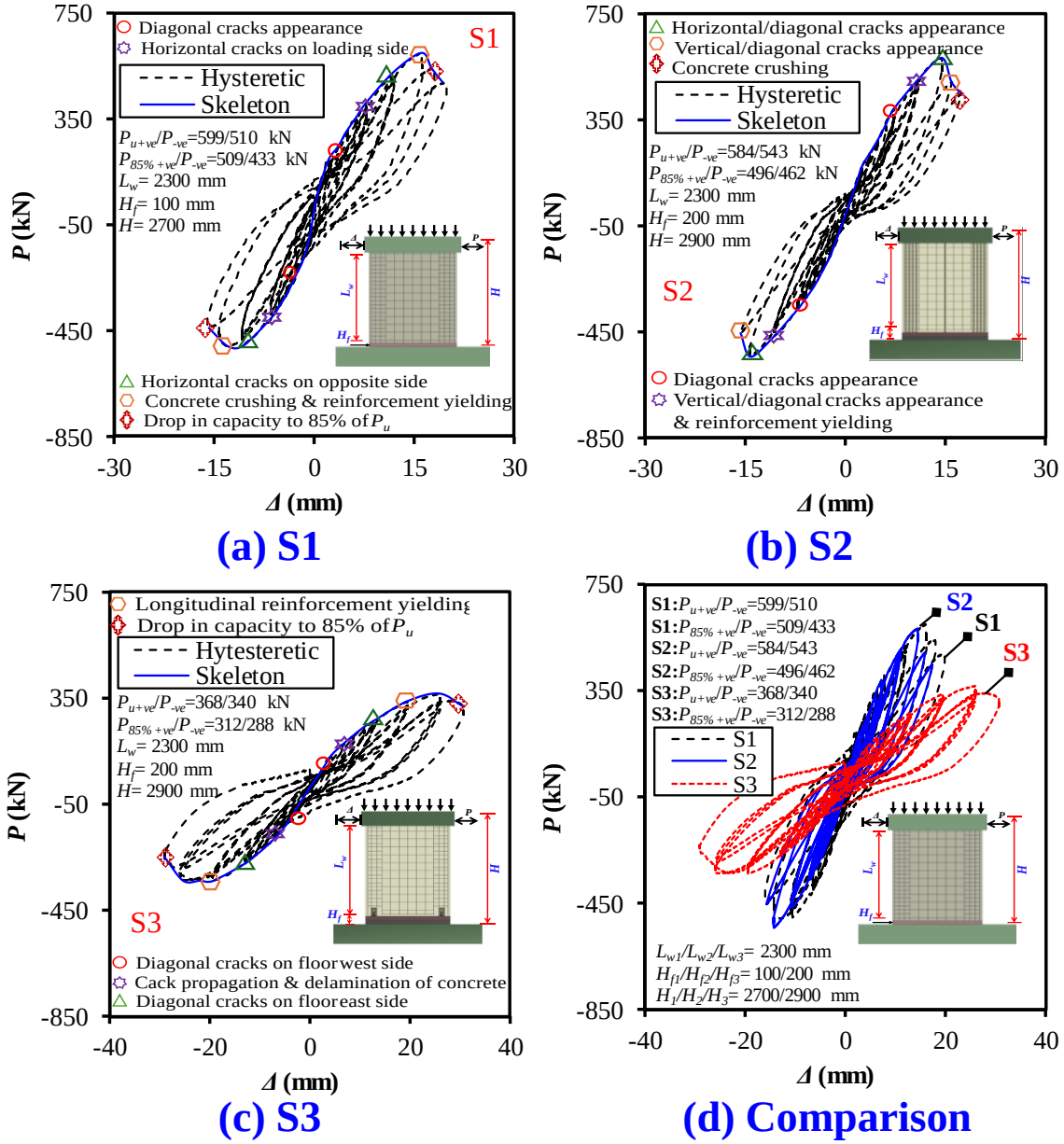


Fig. 12 Hysteretic and skeleton curves of wall specimens

3.7.2.2 Skeleton Curve

The skeleton curves, visualized by connecting the peak points of each loading cycle, depict the strength and deformation characteristics of the walls throughout the loading process (see **Fig. 12a–d**). These curves are divided into three phases: elastic (Stage 1), elastoplastic (Stage 2), and failure or degradation (Stage 3). Key seismic performance parameters—such as yield strength (P_y), yield stiffness (K_e), ultimate capacity (P_u), displacement at ultimate load (Δ_u), inter-story drift ratio at

ultimate displacement (θ_u), pre-ultimate ductility ratio (μ), and post-ultimate ductility index (DI)—
were computed using **Eqn. 2** [54–56]:

$$\theta_u = \frac{\Delta_u}{(L_w + 0.5h_b + h_f)}; K_e = \frac{P_{45\%}}{\Delta_{45\%}}; \mu = \frac{\Delta_u}{\Delta_y}; DI = \frac{\Delta_{85\%}}{\Delta_u} \quad (2)$$

where L_w , h_b , h_f , $P_{45\%}$, $\Delta_{45\%}$, and $\Delta_{85\%}$ signify the height of the wall panel/loading beam/floor
panel, 45% of P_u , and lateral displacement at $P_{45\%}$ and $P_{85\%}$.

In **Stage 1**, the skeleton curves showed apparent linearity, reflecting elastic behavior. P_y and K_e
were derived: S1, S2, and S3 recorded P_y of 249.56 kN, 253.68 kN, and 159.20 kN, respectively—
indicating a 1.6% increase for S2 and 36.2% reduction for S3 relative to S1. Similarly, K_e for S1,
S2, and S3 were 78.23 kN/mm, 59.54 kN/mm, and 26.81 kN/mm, respectively. S2 exhibited a
23.9% fall, and S3 had a 65.7% decrease compared to S1. The increase in P_y and K_e for S2 suggests
better structural integrity, enhancing the wall's ability to resist loads. In contrast, S3's lower P_y and
 K_e can be attributed to the flexible connection, which reduces its capacity compared to S1 and S2.

In **Stage 2**, the skeleton curves shifted into the nonlinear region, indicating that the structure had
entered plastic deformation. P_u , Δ_u , and θ_u were key parameters to analyze the elastoplastic
behavior of walls at this stage. The P_u for S1, S2, and S3 was 554.59 kN, 563.73 kN, and 353.78
kN, with S2 exceeding S1 by 1.6% and S3 falling 36.2% below. S3, unlike S1 and S2, relied on
bolts for load transfer. Loosening of bolts led to localized shear stress concentrations and interface
slip, further exacerbated by unroughened contact surfaces, contributing to its lowest capacity.
Meanwhile, Δ_u for S1, S2, and S3 was 14.50 mm, 14.32 mm, and 22.60 mm, showing S3 had the
highest ductility (+55.9%) but at the cost of strength, indicating better ductility and energy
absorption before failure. According to the JGJ 3–2010 [57], the maximum θ_u for shear walls
should not exceed 1/120. As shown in **Table 4**, S1 and S2 have drift ratios within the limit, with
S1 (S2) at 1/161 (1/187) positive and 1/201 (1/190) negative, yielding a mean of 1/179 (1/189).

However, S3 exhibited a higher drift ratio of 1/104 (positive) and 1/139 (negative), with a mean of 1/119, exceeding the limit —due to interface slip from unroughened surfaces and bolt loosening. In **Stage 3**, the curves declined sharply after attaining ultimate capacity as the structure failed. Pre- and post-ultimate ductility ratios, i.e., μ and DI , became crucial parameters. The DI (μ) values for S1, S2, and S3 were 1.33 (4.51), 1.10 (3.37), and 1.30 (3.80), respectively. S2 exhibited an 18% (25%) lower DI (μ) than S1, indicating better energy dissipation but less displacement before failure. S3 showed a 2% lower DI than S1, but a 16.9% higher μ than S2, though still 15.7% lower than S1. This suggests that S3 exhibited higher deformability, ductility, and energy dissipation capacity due to interface slip, but the sliding remained within acceptable limits, making it suitable for rapid-assembly, short-lifespan structures in low-seismic zones. In contrast, S2 demonstrated stable load transfer, minimal slip, and robust energy dissipation, supporting its use in long-term structures in high-seismic areas.

Table 4 Results of load-displacement responses of S1, S2, and S3 wall-tested specimens

Specimen	Item	Elastic/Stage 1		Plastic/Stage 2			Failure/Stage 3				
		P_y (kN)	K_e (kN/mm)	Δ_u (mm)	P_u (kN)	θ_u (%)	$\Delta_{85\%}$ (mm)	$P_{85\%}$ (kN)	$\theta_{85\%}$ (%)	DI (Ratio)	μ (%)
S1	+ve	270	65	16.1	599	1/161	18.6	509	1/139	1.16	3.91
	-ve	229	91	12.9	510	1/201	19.2	433	1/135	1.50	5.11
	Mean	250	78	14.5	555	1/179	18.9	471	1/137	1.33	4.51
S2	+ve	263	57	14.4	584	1/187	15.9	496	1/170	1.09	3.14
	-ve	245	62	14.2	543	1/190	15.6	462	1/173	1.10	3.60
	Mean	254	60	14.3	564	1/189	15.75	479	1/171	1.10	3.37
S3	+ve	165	28	25.8	368	1/104	30.7	312	1/88	1.19	4.31
	-ve	153	26	19.4	340	1/139	27.4	289	1/99	1.41	3.30
	Mean	159	27	22.6	354	1/119	29.1	300	1/93	1.30	3.80
Specimen	P_t (kN)	$P_{u, mean}$ (kN)	$\frac{P_t}{P_{u, mean}}$								
S1	543	555	0.98								
S2	543	564	0.96								
S3	543	354	1.53								
Mean			1.16								
Cov			0.23								

P_y is the yield load, K_e is the yield stiffness, Δ_u is the displacement at the ultimate load, P_u is the ultimate load, θ_u is the ultimate drift ratio, $\Delta_{85\%}$ is the displacement at 85% of the ultimate load, $P_{85\%}$ is the load at 85% of ultimate capacity, $\theta_{85\%}$ is the drift ratio at 85% load, DI is the post-ultimate ductility index, μ is the pre-ultimate ductility ratio, P_t is the load-carrying capacity of the wall obtained via theoretical model, $P_{u, mean}$ is the mean capacity obtained from the average of positive and negative P_u , and Cov represent coefficient of variance.

3.7.3 Energy Consumption

The area enclosed by the hysteresis loop of a wall represents its energy dissipation capacity, with the energy absorbed during each cycle being proportional to the area of the corresponding loop. Key indices such as energy dissipation capacity, energy consumption coefficient, cumulative energy dissipation, and equivalent hysteresis damping were calculated per **Eqn. 3** to quantify the structure's energy dissipation [58,59]. As shown in **Fig. 13(a)**, energy dissipation for all specimens grew with lateral displacement. While energy consumption was initially minimal, it increased substantially during the second and third cycles. S1 and S2 maintained relatively consistent energy dissipation, but S3 exhibited a marked increase during the later cycles. In **Fig. 13(b)**, the energy dissipation coefficient—representing the ratio of energy dissipated per cycle to total energy absorbed—reveals that S3 provided better energy dissipation than S1 and S2. The energy consumption coefficient for S3 was consistently higher across cycles, peaking at 0.264385, compared to 0.149414 for S2 and 0.166423 for S1. **Fig. 13(c)** shows a gradual increase in energy absorption for S1 and S2, while S3 exhibited a steeper rise, particularly in later cycles, indicating superior energy dissipation during cyclic loading. S3 had the highest cumulative energy of 39,748.86 J, a 9% increase over S1's 36,632.26 J and a 95% increase over S2's 20,337.46 J. The damping ratio for S3 was the highest at 13.8%, outperforming S1 at 12.3% and S2 at 12.0%, underscoring its superior energy dissipation, as depicted in **Fig. 13(d)**. Moreover, the hysteresis curves in **Fig. 12(a-d)** revealed that while the loop areas were similar at lower displacements, they were fuller and more rounded in S3, indicating enhanced ductility and energy dissipation capacity compared to S1 and S2.

$$E_i = \frac{S_{ABC} + S_{CDA}}{S_{DOBE} + S_{DODF}}; \eta_i = \frac{E_i}{\sum_{i=1}^n E_i}; E_{cum} = \sum_{i=1}^n E_i; \xi = \frac{S_{ABCD}}{2\pi(S_{DOBE} + S_{DODF})} \quad (3)$$

where E_i , $S_{ABC}/S_{CDA}/S_{\Delta OBE}/S_{\Delta ODF}$, η_i , E_{cum} , and ξ represent energy dissipation capacity during the i^{th} cycle, energy dissipated geometric areas of the hysteresis loop, energy consumption coefficient for the i^{th} cycle, cumulative energy dissipation, and the equivalent hysteresis damping ratio.

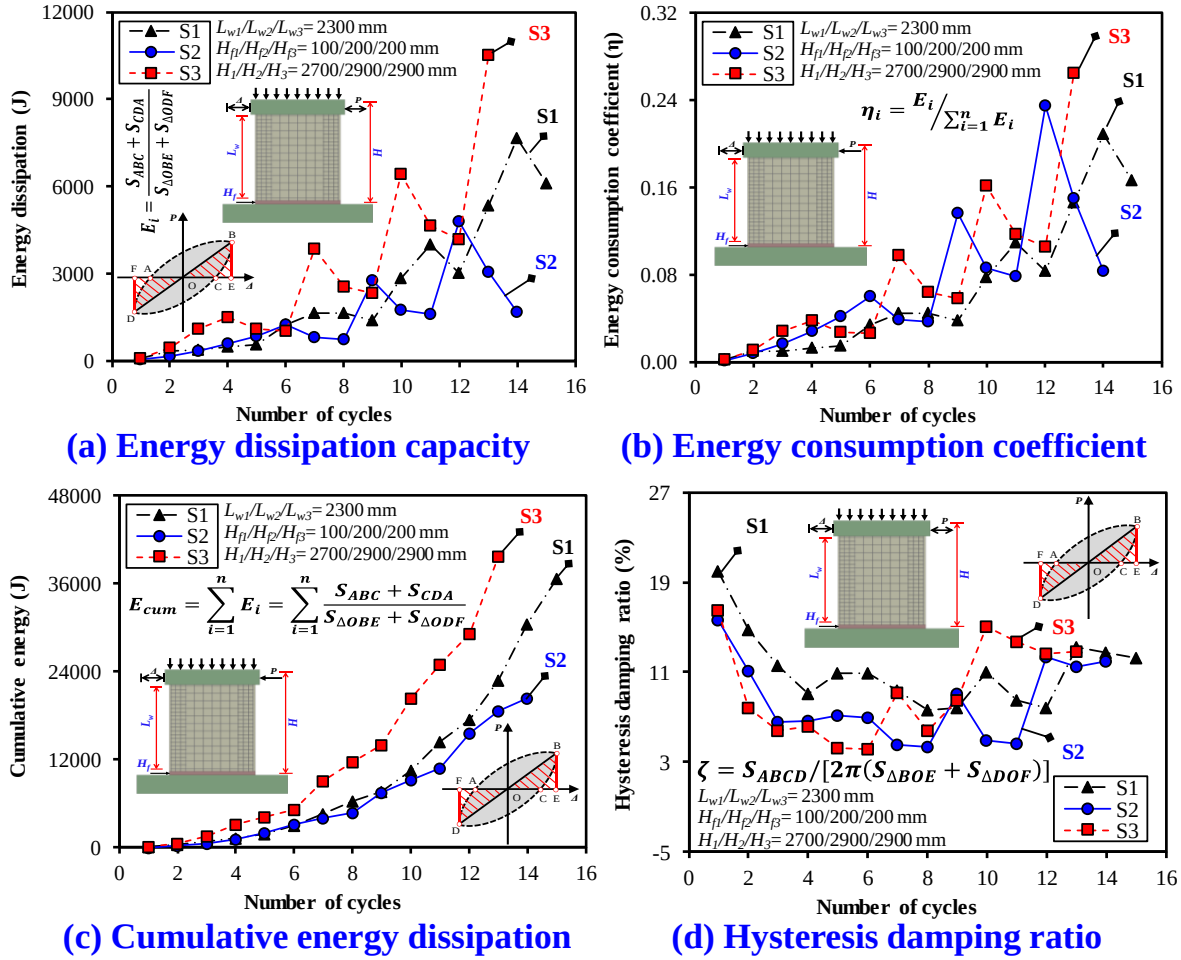


Fig. 13 Energy consumption capacity of wall specimens

3.7.4 Strength Degradation

Strength degradation reflects the gradual decline in a specimen's load-carrying capacity during cyclic loading at constant drift levels. It is quantified by the strength degradation coefficient $\lambda_i = P_i^3/P_i^1$, where P_i^1 and P_i^3 are the peak loads in the first and third cycles at the i^{th} drift ratio [58,59]. As shown in **Fig. 14(a)**, all specimens exhibited decreasing λ_i with increasing lateral displacement. Prior to yielding, fluctuations were more pronounced as the specimens were still in the

strengthening phase. After yielding, the coefficients stabilized within the range of 0.8 to 1.5 (both directions), showing limited variation among specimens. While connection type had minimal influence on pre-yield behavior, post-yield degradation varied. Notably, S3 showed a slower rate of strength degradation than S1 and S2, indicating its greater deformation tolerance due to interface flexibility—albeit at the cost of reduced initial stiffness and load-carrying capacity. In contrast, S2 retained a strength profile closer to S1, benefiting from efficient force transfer through its through-reinforced grouted IMC. These results suggest that although S3 is unsuitable for high-strength or permanent structures, its gradual degradation and deformation capacity may suit temporary or low-risk modular applications.

3.7.5 Stiffness Degradation

Stiffness degradation refers to reduced lateral resistance as wall displacement increases under cyclic loading. It is evaluated using secant stiffness $K_j = \sum_{i=1}^n P_j^i / \sum_{i=1}^n \Delta_j^i$, where P_j^i and Δ_j^i are the force and displacement at the j^{th} drift level [59,60]. As shown in **Fig. 14(b)**, all specimens initially exhibited high stiffness during the elastic phase. Upon crack initiation, stiffness declined rapidly. With continued cycling, damage accumulated, and the degradation rate slowed, leading to flatter curves. S1 and S2 followed similar stiffness degradation paths and maintained relatively higher secant stiffness across all drift levels. In contrast, S3 consistently showed lower stiffness, attributed to its bolted connection permitting greater interface movement. While this reduced initial resistance, it enhanced ductility and energy dissipation. This trade-off underscores the need to select connection types based on expected seismic performance and service life: S3 offers deformability at the expense of stiffness, while S2 provides a more balanced combination of strength, stiffness, and long-term performance.

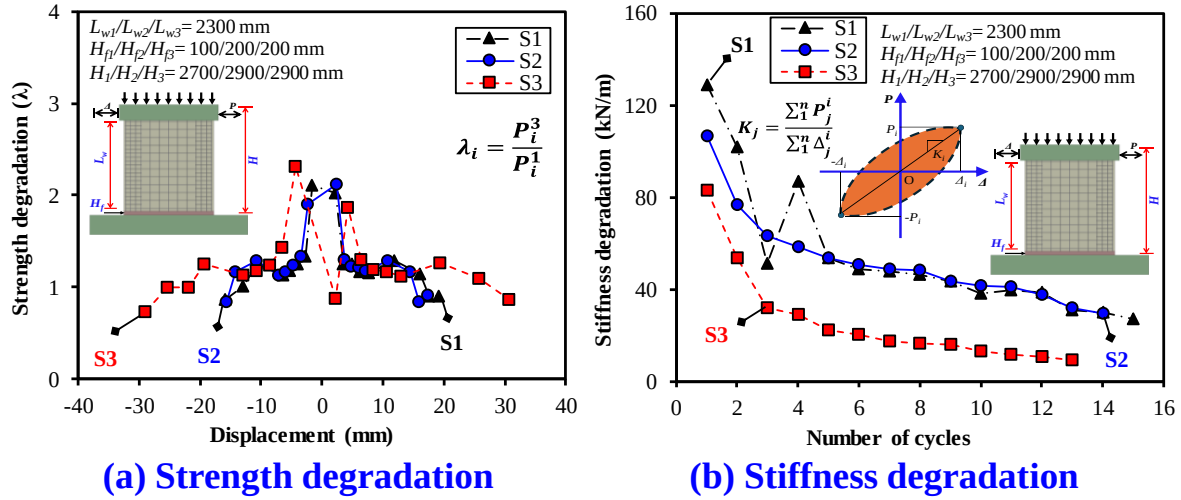


Fig. 14 Strength and stiffness degradation of wall specimens

3.7.6 Strain Analysis

Strain data from embedded gauges (**Figs. 5(a-c)** and **7(c-d)**) provide detailed insights into reinforcement behavior during cyclic loading. Readings were analyzed at each specimen's yield, peak, and failure stages (**Figs. 15(a-c)**). As loading progressed, strain in the longitudinal rebars increased steadily, indicating the transition from elastic to plastic behavior. Yielding was progressive, with direction-specific partial recovery observed at failure stages due to unloading or reverse cycles. S1 and S2 displayed comparable strain patterns, reflecting similar wall behavior and connection stiffness. In contrast, S3 exhibited lower strain amplitudes, especially in early cycles, which was attributed to reduced force transfer through the bolted interface. This interface flexibility delayed strain accumulation and mitigated stress localization in the wall, preventing early failure. This observation aligns with experimental findings (**Fig. 11**), where deformation was concentrated at floor-slab interfaces and wall damage remained limited. These results confirm that the S3 bolted design allows controlled deformation with lower internal stress—which is beneficial for temporary or reusable modular structures. Conversely, S2's continuously reinforced grouted connection ensures complete stress transfer and better seismic resilience, supporting its application in high-seismic, long-service-life environments.

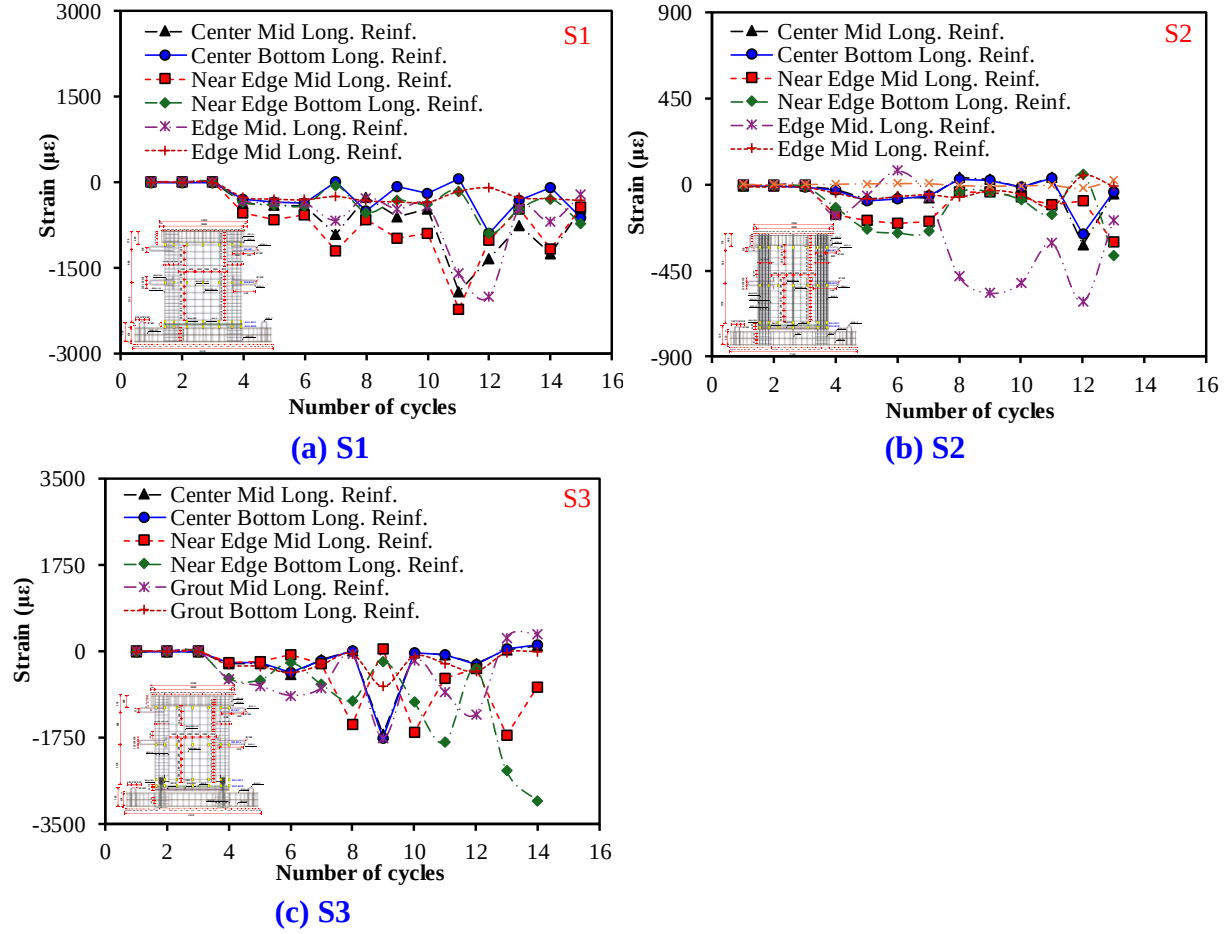


Fig. 15 Strain distribution in wall specimens

4 Numerical Study on Seismic Performance of PVMS Walls

Experimental testing provides valuable insight into the seismic behavior of PVMS walls but does not fully capture internal failure mechanisms, steel-concrete interaction, or parameter sensitivities. To address these limitations, a 3D nonlinear FEM was developed using experimental failure modes and load-displacement data for validation. This model supports a deeper investigation of elastoplastic seismic response, providing a reliable reference for future design refinements under similar conditions.

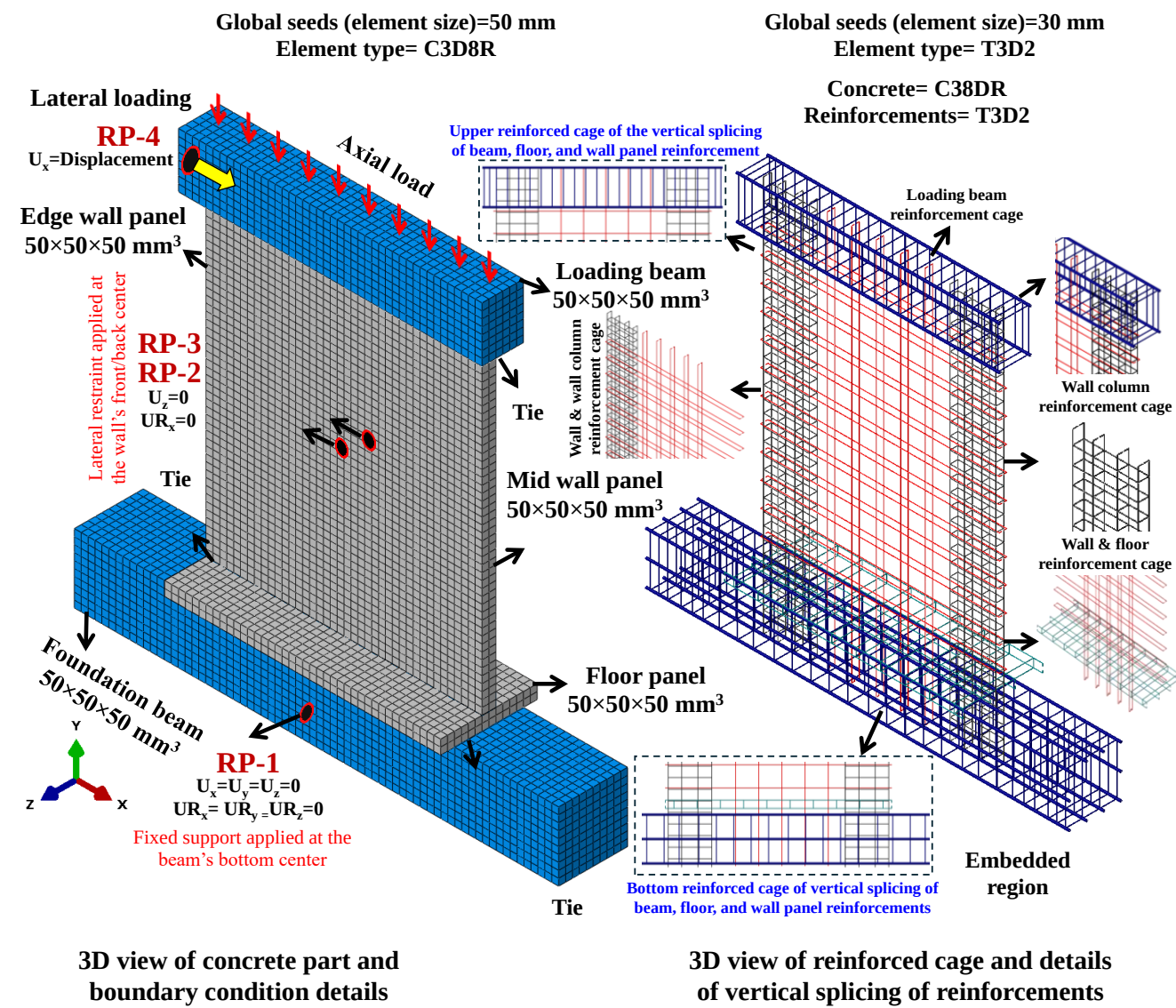
4.1 Development of Finite Element Mesh Model

A detailed FEM of precast walls was developed in ABAQUS 2023 [61], with ABAQUS/CAE used for pre-processing and ABAQUS/Standard for analysis. The model included all critical

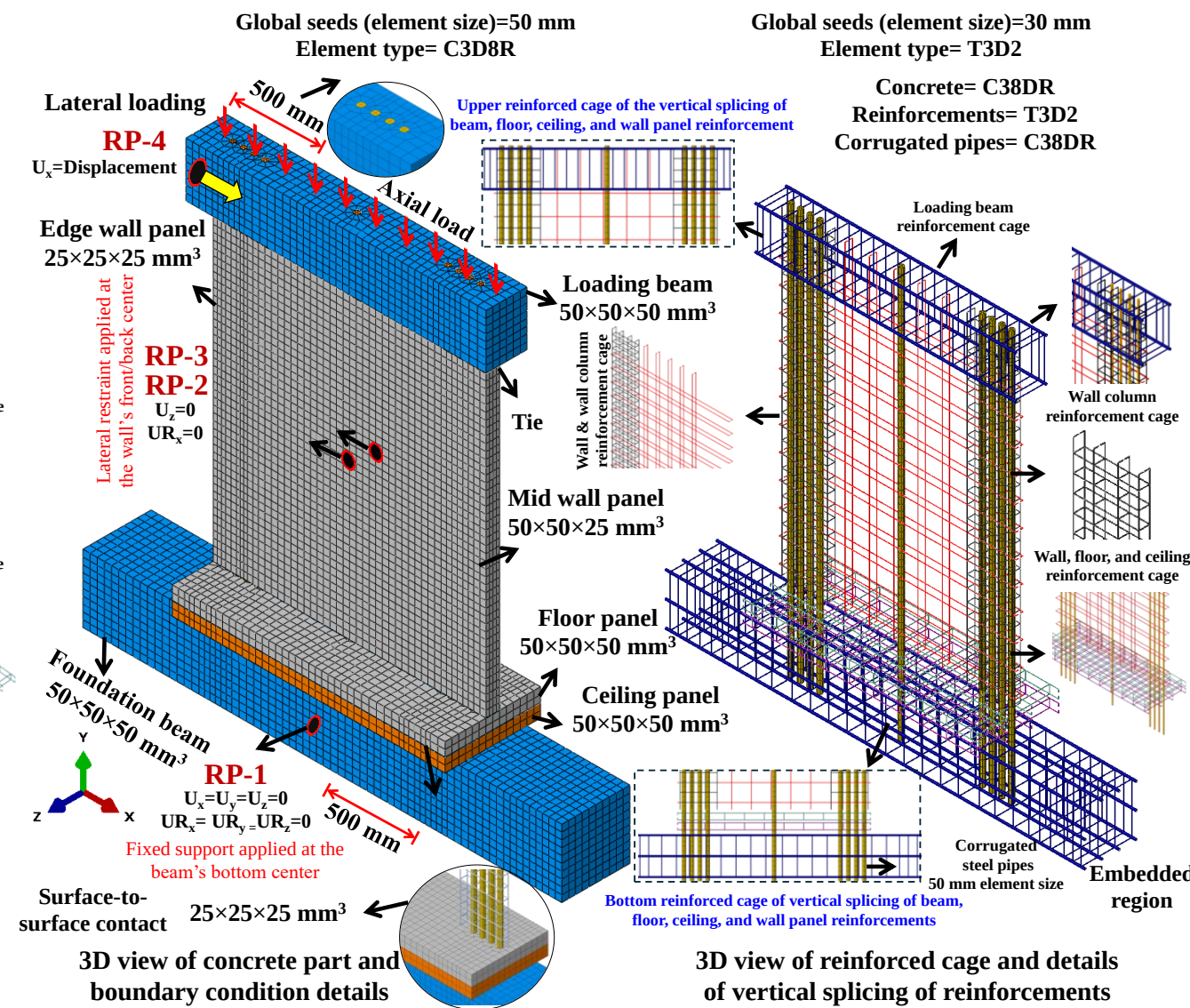
components—RC wall, loading beam, floor and ceiling slabs, foundation beam, mortar, corrugated pipes, bolts, and nuts—as shown in **Fig. 16(a–c)**. Concrete, mortar, bolts, nuts, and pipes were modeled using 8-node linear brick elements with reduced integration (C3D8R). Reinforcement cages and stirrups were simulated with 2-node truss elements (T3D2) [58]. A mesh sensitivity analysis was conducted to ensure accuracy. Concrete elements and steel components were meshed at 50 mm, while reinforcement cages used 30 mm mesh. For wall panels: S1 used a uniform $50 \times 50 \times 50 \text{ mm}^3$ mesh, S2 applied a refined $25 \times 25 \times 25 \text{ mm}^3$ mesh at the edge regions (first 500 mm) and $50 \times 50 \times 25 \text{ mm}^3$ mesh centrally, and S3 employed $15 \times 50 \times 15 \text{ mm}^3$ mesh near the bolted IMC zones (first 60 mm) and $50 \times 50 \times 15 \text{ mm}^3$ mesh elsewhere in the mid-region.

4.2 Material Modeling

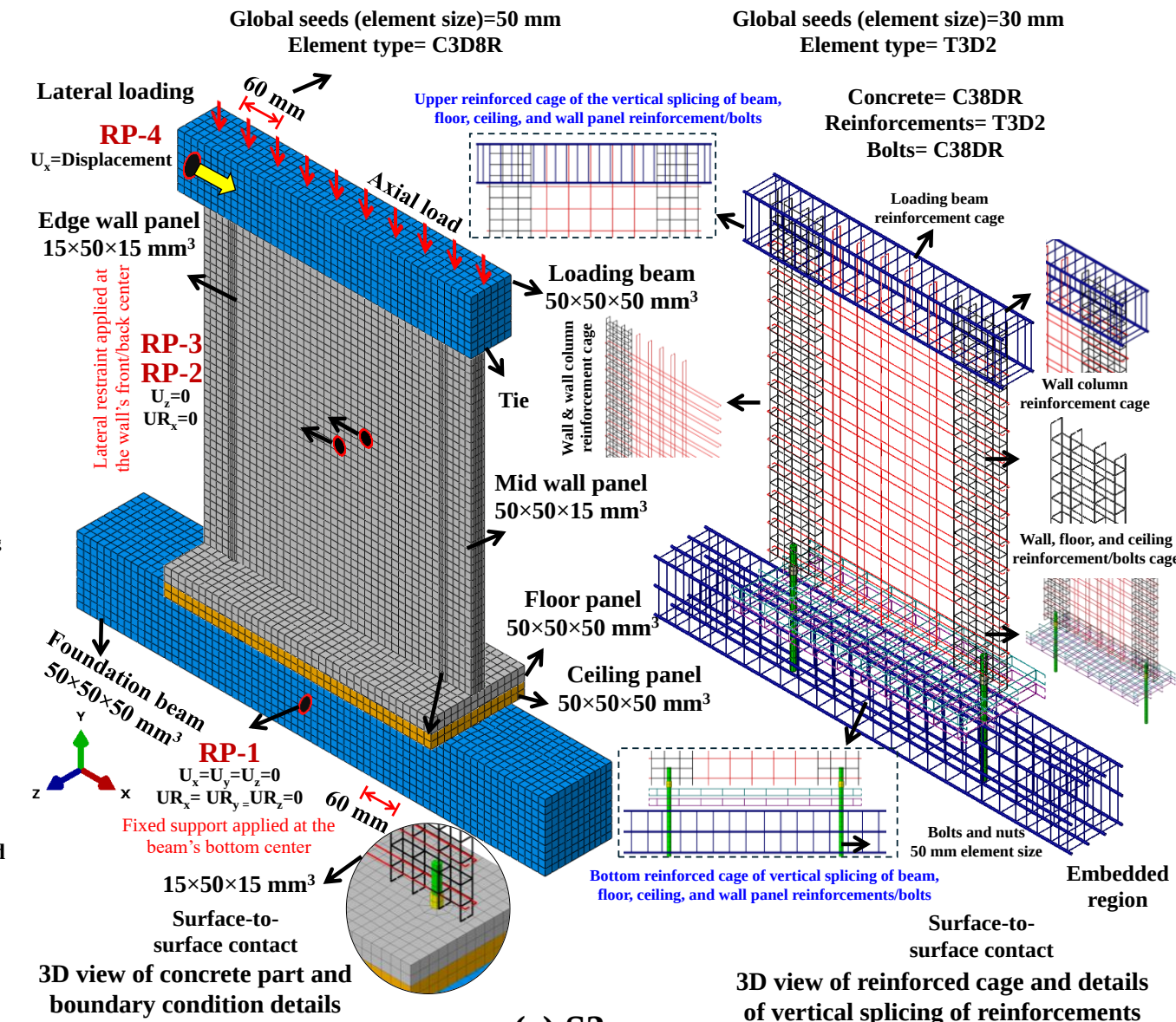
Concrete was modeled using the Concrete Damage Plasticity (CDP) model to represent its nonlinear elastoplastic and damage behavior. Material properties followed GB 50010-2010 [45], using uniaxial stress-strain curves and tension-compression damage factors (**Fig. 17a–b**). CDP parameters included: dilation angle = 30%, eccentricity = 0.1, viscosity = 0.0005, $\sigma_{b0}/\sigma_{c0} = 1.16$, and $K_c = 2/3$. Steel reinforcements and bolts were defined using a bilinear elastic-plastic model with von Mises yield and kinematic hardening (**Fig. 17c**). After yielding, the modulus of elasticity was reduced to $E_s' = 0.01E_s$, with a linear stress-strain response in both tension and compression. A Poisson's ratio of 0.3 was used, assuming that the yield surface coincides with the plastic potential surface.



(a) S1



(b) S2



(c) S3

Fig. 16 Finite element model establishment for wall specimens, including details of mesh, contact, and boundary conditions.

4.3 Contacts and Boundary Constraints

Steel reinforcements in walls, slabs, and beams were modeled as embedded truss elements within their respective concrete domains using the "Embedded Region" constraint. This ensured effective bond behavior without explicitly modeling rebar-concrete slip. For S2, through-type reinforcement and corrugated pipes were modeled using surface-to-surface contact with surrounding concrete components (wall, slab, and beam). For S3, bolts and nuts were modeled explicitly, while bolt pipes, boxes, and plates were omitted to simplify computation. Bolt pretension was applied via the torque-preload formula $T=K \times d \times P$ [62], using a torque coefficient $K = 0.2$, bolt diameter $d = 30$ mm, and resulting pretension force $P = 175$ kN (torque $T = 1050$ Nm). All bolt-concrete and bolt-nut interfaces used "hard contact" in the normal direction and "finite sliding" with a friction coefficient of 0.6 tangentially. Although minor slip effects were neglected, the penalty-based contact formulation and bolt pretension realistically simulated frictional clamping and force transfer, aligning well with experimental performance [63]. For S1, which was monolithically cast, rigid "Tie Constraints" were applied between the wall and adjoining panels (loading beam, slab, and foundation). For S2 and S3, interface contacts (wall-to-slab, slab-to-foundation) were modeled as surface-to-surface with "hard contact" and finite sliding (friction coefficient = 0.5) to account for interfacial friction. The contact between the loading beam and wall (top surface) remained tied due to simultaneous casting. Concrete was treated as the host material for embedded regions, and load-bearing interfaces (e.g., mortar layers) used Tie constraints to simulate cast-in-place bonding. The loading and foundation beams were defined as master surfaces due to their larger contact areas. To replicate boundary and load conditions from the experiment, four reference points (RP-1 to RP-4) were defined at key locations: RP-1 at the center bottom of the foundation, RP-2 and RP-3 at mid-height wall positions, and RP-4 at the top loading beam center. Coupling constraints linked

these to the surrounding surfaces to apply boundary conditions. The foundation was fully restrained, while lateral supports restricted out-of-plane displacement and rotation. Axial loads were applied on the top surface of the loading beam as a pressure load, while lateral cyclic loading was applied through RP-4 to simulate actuator forces.

4.4 Validation of Finite Element Modeling

To assess the accuracy of the FEM approach, numerical simulations of S1, S2, and S3 were conducted and compared against experimental results. The load-displacement responses, crack formation zones, and failure modes predicted by the models were validated against test outcomes. This verification confirmed the suitability of the adopted mesh schemes, contact definitions, material models, and boundary conditions in capturing global and localized structural behavior. The models demonstrated good agreement in strength, stiffness, and displacement capacity, ensuring that the FEM can reliably support further parametric studies and theoretical design model development.

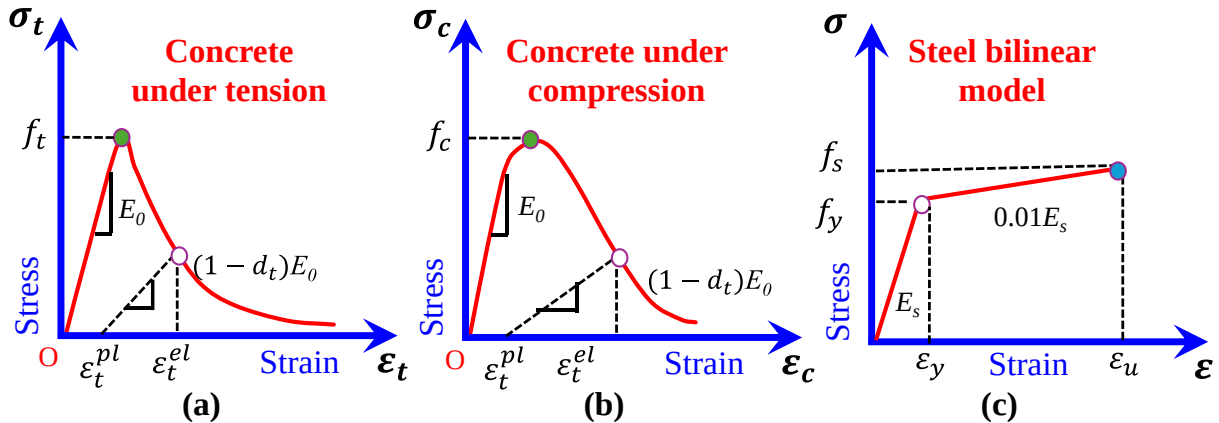


Fig. 17 Graphical representation of the CPD model (a and b) and the bilinear model of steel (c)

4.4.1 Comparison of Failure Modes

Fig. 18(a-c) presents a comprehensive comparison between the experimental failure modes and those predicted by FEMs for S1, S2, and S3. The comparison includes observed crack patterns,

FEM-based concrete tensile damage contours, plastic equivalent strain (PEEQ) distributions, and rebar stress responses. In all cases, the simulations effectively captured the primary failure characteristics observed during testing, including the localization of cracking, regions of high stress, and overall deformation patterns. All three specimens displayed bending-dominated failure behavior, with cracking concentrated at the wall–floor interface and extending diagonally from the wall corners. The formation of horizontal cracks at the base, cover spalling, and localized concrete crushing were consistently observed in both experimental and numerical results. Rebar stress contours confirmed yielding at the tension side of the wall, particularly at mid-height and near the bottom edge, consistent with the development of a plastic hinge region. PEEQ contour plots revealed distinctions in plastic strain evolution among the specimens. In S1, PEEQ values reached a maximum of 0.1196, localized at the base and mid-height regions. This confirmed a well-developed plastic hinge with a balanced tension-compression response indicative of global flexural deformation. In S2, the maximum PEEQ was much lower, at 0.00196, and remained confined to the grouted interface and lower wall zone. The limited plastic strain reflects the enhanced confinement and crack resistance the through-reinforced grouted IMC provides, supporting the stable crack pattern and minimal wall deformation observed experimentally. S3 exhibited the highest PEEQ value, reaching 0.2329. Plastic strain distribution was concentrated along the slab–wall interface and around the bolt locations, consistent with localized slip and joint-level deformation. While the wall remained largely undamaged, the deformation was accommodated by relative movement at the connection, allowing controlled energy dissipation without widespread internal damage.

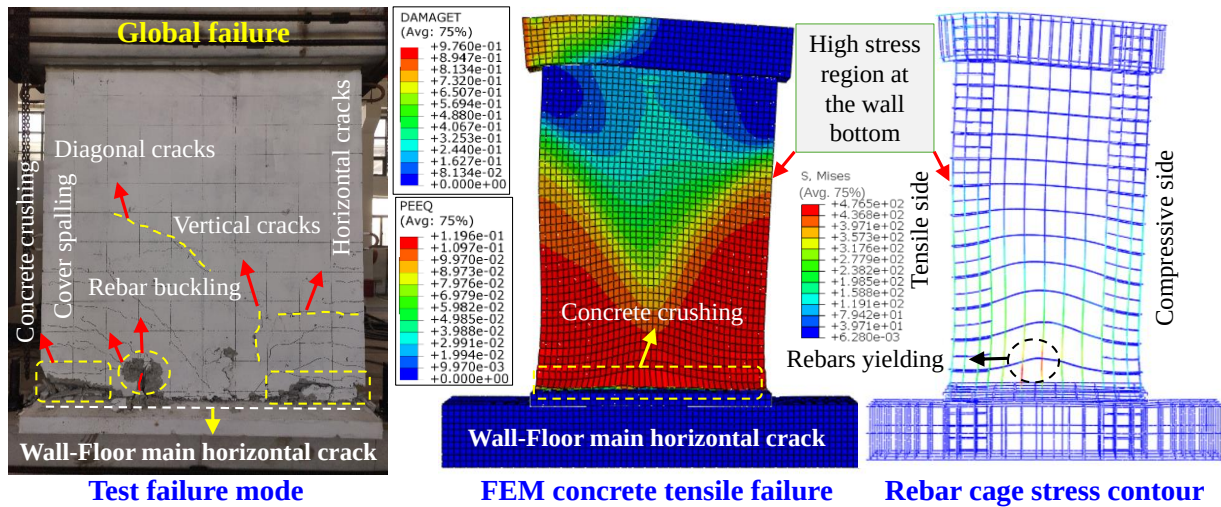
Although concrete tensile damage contours were broadly similar across all specimens, with maximum values approaching 0.976, the PEEQ contours highlighted apparent differences in

plastic deformation mechanisms. In S2, the strain was efficiently transferred and confined, while S3 showed distributed plasticity through its flexible bolted IMC. These differences reflect the distinct mechanical behavior of each connection type under lateral cyclic loading. The FEM results for S1 and S2 accurately identified high-stress zones from the mid-height to the wall base, with stress diminishing toward the top. For S3, the FEM captured the primary stress concentration zones but could not replicate minor interface slips due to idealized surface modeling. Nonetheless, the predicted failure modes—characterized by diagonal cracking, interface gapping, and local crushing—strongly agree with the test outcomes, validating the effectiveness of the numerical models in simulating the seismic response of PVMS walls.

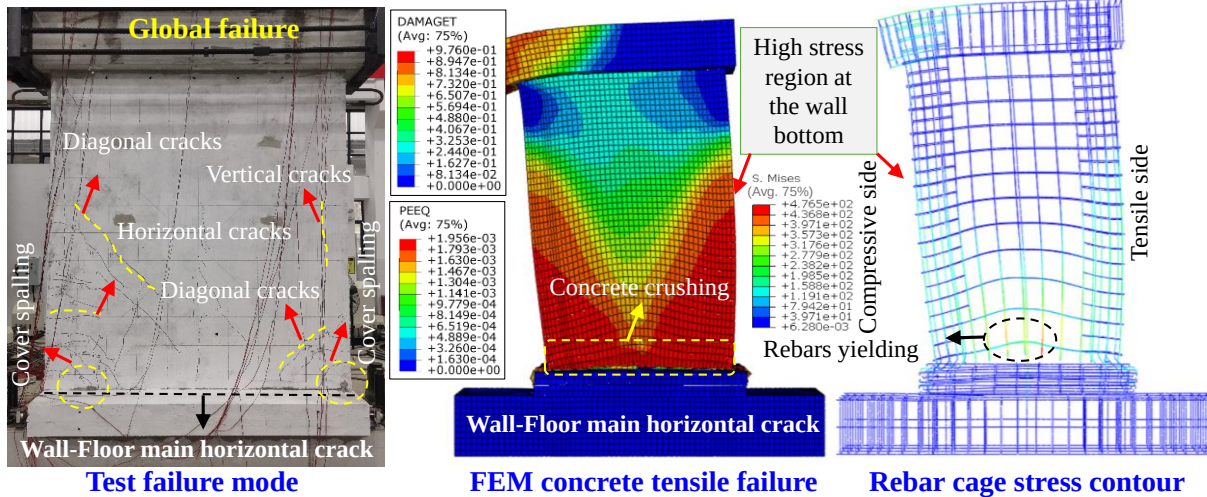
Table 5 Comparison of characteristic parameters between tests and FEM results for wall

Item	Loading direction	$P_{u, Test}$ (kN)	$P_{u, FEM}$ (kN)	$\frac{P_{u, Test}}{P_{u, FEM}}$	$P_{y, Test}$ (kN)	$P_{y, FEM}$ (kN)	$\frac{P_{y, Test}}{P_{y, FEM}}$	$K_{e, Test}$ (kN/mm)	$K_{e, FEM}$ (kN/mm)	$\frac{K_{e, Test}}{K_{e, FEM}}$
S1	+ve	599.40	598.57	1.00	269.73	269.36	1.00	65.42	92.60	0.71
	-ve	509.77	569.37	0.90	229.40	256.21	0.90	91.03	131.17	0.69
	Mean	554.59	583.97	0.95	249.56	262.79	0.95	78.23	111.88	0.70
S2	+ve	583.97	618.47	0.94	262.79	278.31	0.94	57.11	102.79	0.56
	-ve	543.48	580.28	0.94	244.57	261.13	0.94	61.98	125.31	0.49
	Mean	563.73	599.38	0.94	253.68	269.72	0.94	59.54	114.05	0.52
S3	+ve	367.77	380.02	0.97	165.50	171.01	0.97	27.65	40.57	0.68
	-ve	339.79	350.03	0.97	152.90	157.51	0.97	25.98	51.85	0.50
	Mean	353.78	365.03	0.97	159.20	164.26	0.97	26.81	40.21	0.67
Item	Loading direction	$\Delta_{u, Test}$ (mm)	$\Delta_{u, FEM}$ (mm)	$\frac{\Delta_{u, Test}}{\Delta_{u, FEM}}$	DI_{Test} (Ratio)	DI_{FEM} (Ratio)	$\frac{DI_{Test}}{DI_{FEM}}$	μ_{Test} (mm)	μ_{FEM} (mm)	$\frac{\mu_{Test}}{\mu_{FEM}}$
S1	+ve	16.11	19.69	0.82	1.16	1.57	0.74	3.91	6.77	0.58
	-ve	12.89	15.41	0.84	1.50	0.46	3.27	5.11	7.89	0.65
	Mean	14.50	17.55	0.83	1.33	1.01	1.32	4.51	7.33	0.61
S2	+ve	14.44	15.95	0.91	1.10	1.31	0.84	3.14	5.89	0.53
	-ve	14.20	13.21	1.08	1.10	1.31	0.84	3.60	6.34	0.57
	Mean	14.32	14.58	0.98	1.10	1.31	0.84	3.37	6.11	0.55
S3	+ve	25.81	20.84	1.24	1.19	1.75	0.68	4.31	4.94	0.87
	-ve	19.39	18.50	1.05	1.41	1.80	0.79	3.29	6.09	0.54
	Mean	22.60	19.67	1.15	1.30	1.77	0.73	3.80	5.52	0.69

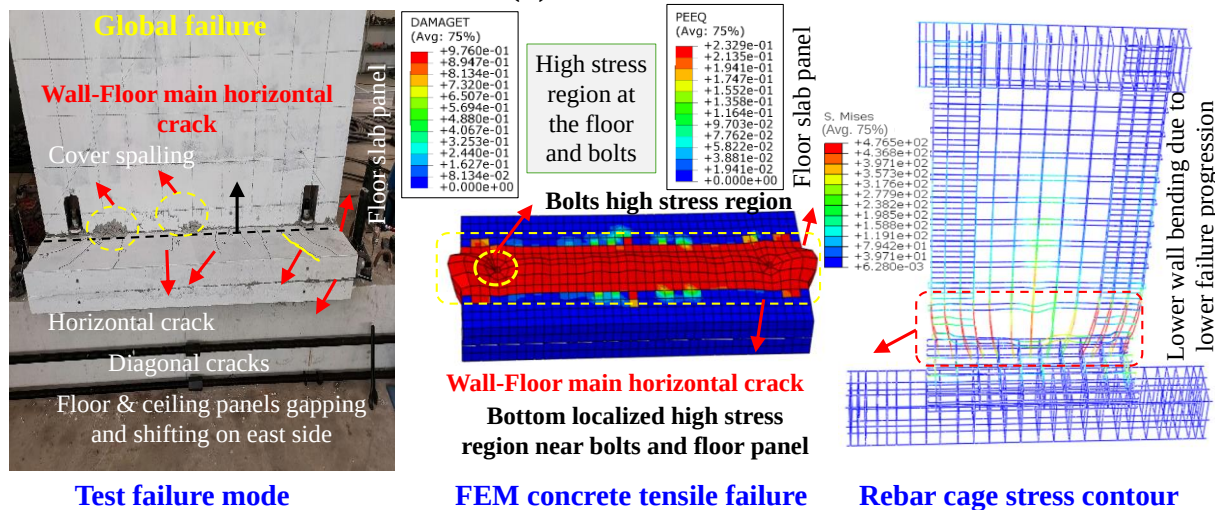
Note: $P_{u, Test}$, $P_{u, FEM}$, $P_{y, Test}$, $P_{y, FEM}$, $K_{e, Test}$, $K_{e, FEM}$, $\Delta_{u, Test}$, $\Delta_{u, FEM}$, DI_{Test} , DI_{FEM} , μ_{Test} , and μ_{FEM} represent lateral ultimate capacity, lateral yield capacity, yield stiffness, displacement at ultimate load, post-ultimate ductility index, and pre-ultimate ductility ratio for the experimental tests and FEM simulations, respectively.



(a) S1: Test vs FEM



(b) S2: Test vs FEM



(c) S3: Test vs FEM

Fig. 18 Comparison of failure modes between experimental tests and FEMs for S1, S2, and S3

4.4.2 Comparison of Load-Displacement Curves

Fig. 19(a-c) compares the hysteresis curves obtained from both experiments and FEM simulations for S1 to S3. In all cases, the simulated and test curves align well during the initial elastic phase, confirming the FEM's accuracy in capturing early-stage structural behavior. However, as displacement increases, the FEM curves display slightly higher stiffness than observed experimentally, particularly in post-yield stages. **Fig. 19(d-f)** and **Table 5** compare skeleton curves and key test-to-FE ratios— P_u , P_y , K_e , Δ_u , DI , and μ —averaged across both positive and negative loading cycles for S1, S2, and S3. The average P_u/P_y ratios for S1 (0.95), S2 (0.94), and S3 (0.97) show a strong match between FEM and experimental data, with minimal variation (Cov = 0.01). This affirms the FEM's capacity to predict load-bearing limits with slight conservative tendencies. K_e was overestimated in all cases—particularly for S2—where the test-to-FE ratios were 0.70 (S1), 0.53 (S2), and 0.59 (S3), averaging 0.61 (Cov = 0.12). For Δ_u , S1 and S2 were underestimated (0.83 and 0.99), while S3 was slightly overestimated (1.14). This reflects the influence of connection flexibility in S3, where FEM captured its enhanced ductility, though conservative estimates for S1–S2 suggest the models missed some plasticity effects. Ductility metrics showed greater discrepancies: DI values (2.00 for S1, 0.84 for S2, and 0.73 for S3) exhibited the most variability (Cov = 0.48), implying that the FEMs need refinement in post-peak damage modeling. The μ values were more stable with S1 (S2 and S3) at 0.61 (0.55 and 0.71), yielding a Cov of 0.10, confirming that pre-ultimate ductility is reliably predicted. Overall, FEM predictions fell within an acceptable margin of error for structural applications: average errors were 4.9% (P_u), 3.3% (Δ_u), 4.9% (P_y), and 10.6% (DI). The most considerable divergence occurred in K_e , with a 69% overestimation. The discrepancies observed can be primarily attributed to simplifications in the FEM material model, idealized boundary conditions, limitations of the CPD

model, minor imperfections in specimen manufacturing, assembly tolerances, and differences in surface roughness between the test and numerical models. Despite these simplifications, the FEMs successfully reproduced the overall load-displacement behavior of all specimens and offered a reliable platform for parametric and design studies.

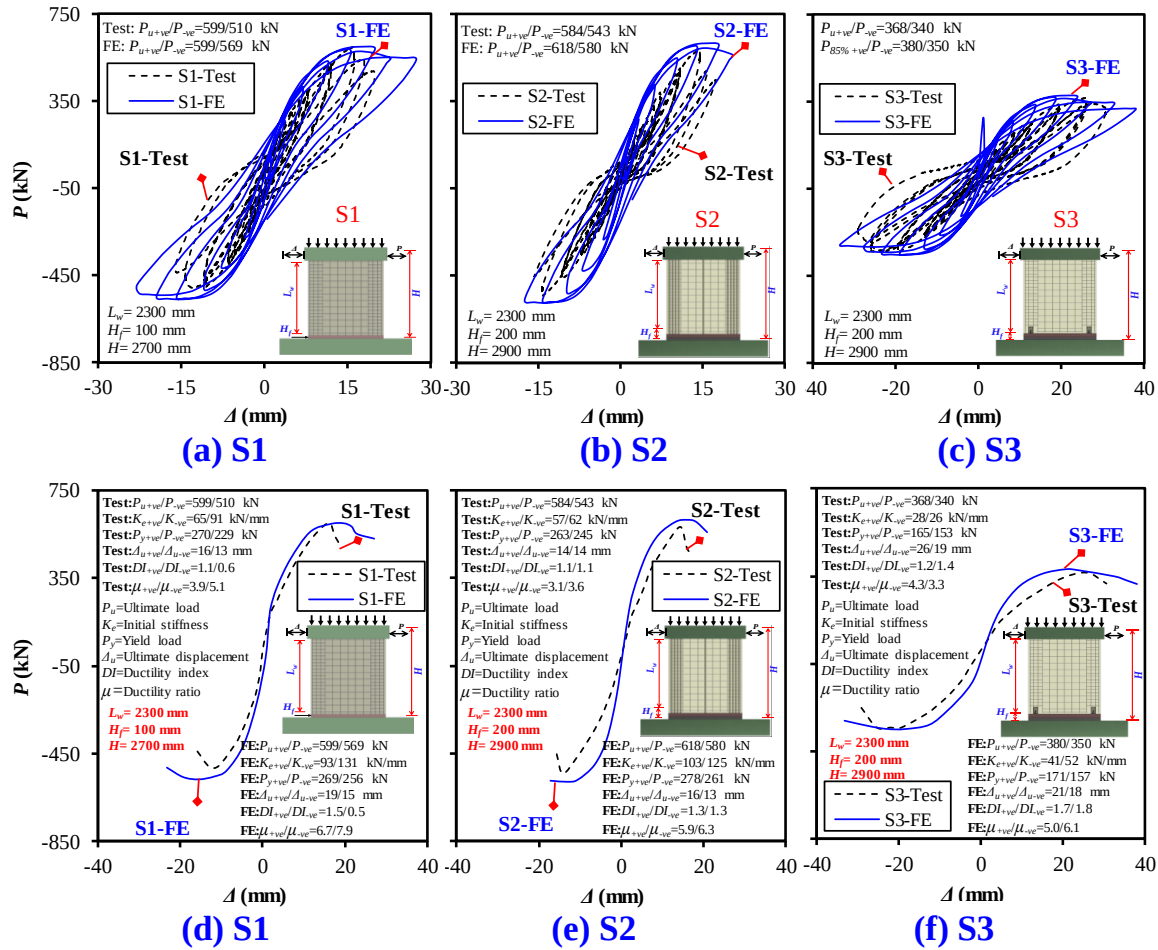


Fig. 19 Comparison of hysteresis and skeleton curves between tests and FEMs for S1, S2, and S3

4.4.3 Crack Propagation

The CPD model used in ABAQUS does not render discrete cracks but instead represents damage through contour plots based on equivalent plastic strain. **Fig. 20(a–c)** visualizes crack evolution in S1 at three critical stages: initial cracking, yielding, and failure. At 2.17 mm displacement, a local tension zone forms at the lower right corner of the wall, marking the first visible crack. As the load

increases to 12.47 mm, this tensile zone propagates upward and laterally, forming new cracks on the opposite corner. By 15.89 mm, the crack network has stabilized, but the plastic damage continues expanding, indicating material degradation rather than new crack initiation. This simulated propagation path mirrors the experimental/FEM crack maps in **Figs. 9** and **18(a)**, confirming that the FEM effectively replicates both the sequence and pattern of failure. Although individual microcracks are not explicitly shown, the gradual transition from localized tension to widespread crushing is well captured. PEEQ contours in **Fig. 18(a)** supplement this by highlighting where irreversible plastic strains accumulate, linking internal mechanics to external crack development observed in physical tests.

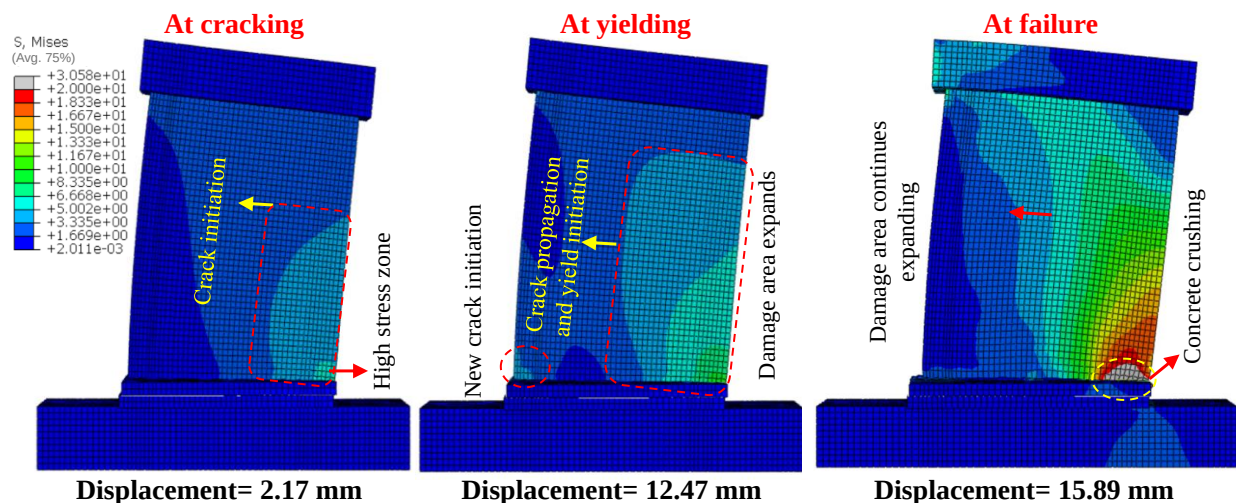


Fig. 20 Stress distribution contour plot for S1 FEM analysis

5 Parametric Analysis

Following validation of the S2 and S3 FEMs against experimental results, a parametric analysis was conducted to assess the influence of critical design parameters on the seismic performance of vertical splice joints in PVMS walls. All models retained consistent material properties, boundary conditions, and mesh refinement to ensure reliability and comparability.

5.1 Walls with Through-Reinforced Grouted Connections

5.1.1 Effect of Number of Reinforcement at Column Edges

Figs. 21(a), 22(a), and 23(a) show that increasing edge reinforcement count (#3, #4, #5) enhances stiffness and load-carrying capacity. P_y increased from 242 kN to 270 kN and 289 kN, and P_u from 539 kN (#3) to 599 kN (#4) and 643 kN—a rise of 19.3%. K_e also improved, with a 48.7% increase from #3 (113 kN/mm) to #4 (114 kN/mm) and #5 (168 kN/mm). However, the post-peak behavior showed a trade-off. DI dropped by 16.4% from #3 to #4, then slightly increased (3.1%) with #5. Similarly, μ and Δ_u exhibited minor variations. From #3 to #4, μ increased by 2.2% and Δ_u by 14.2%, whereas both values decreased by 4.5% and 15.5% from #3 to #5. The results highlight a performance threshold: while more reinforcement enhances strength, it may reduce ductility and deformation capacity, underscoring the need for balance in design.

5.1.2 Effect of Diameter of Reinforcement at Column Edges

As shown in **Figs. 21(b), 22(b), and 23(b)**, increasing bar diameter (16 mm → 18 mm → 20 mm) also improved strength and stiffness. P_y and P_u rose from 252 kN and 560 kN (16 mm) to 270 kN and 599 kN (18 mm) and 289 kN and 643 kN (20 mm), while K_e increased by 31.9% from 16 to 20 mm. DI decreased with thicker bars (−19% for 16 mm → 18 mm), stabilizing thereafter. Interestingly, both Δ_u and μ improved, with μ (Δ_u) rising 20.5% (27.4%) (16→18 mm) and 22.2% (10.2%) (16→20 mm). Unlike the reinforcement count, larger diameters preserved pre-ultimate ductility while limiting post-ultimate deformation, offering a favorable stiffness–ductility trade-off at higher diameters.

5.1.3 Effect of Height-to-Width Ratio of Wall Panel

Varying the wall aspect ratio (Height/Width = 1.05, 1.15, 1.25) while adjusting the wall width (2190 mm to 2000 mm and 1840 mm) impacted seismic performance, as shown in **Figs. 21(c), 22(c), and 23(c)**. Higher ratios reduced P_y (285 kN → 270 kN → 259 kN) and P_u (633 kN → 599 kN → 575 kN), with K_e dropping sharply from 269 kN/mm to 114 kN/mm and 137 kN/mm. The

most significant stiffness loss occurred between 1.05 and 1.15 (−57.5%). DI initially declined (21.4%), then slightly recovered at 1.25 (1.5%). Although Δ_u increased significantly (84.5% for 1.15, 65.2% for 1.25), μ steadily decreased (16.7% for 1.15, 8.6% for 1.25), indicating that greater wall slenderness enhances flexibility but compromises ductility and load resistance. These results suggest diminishing returns in seismic performance beyond a critical height-to-width ratio.

5.2 Walls with Fully-Bolted Connections

5.2.1 Effect of Number of Bolts

Figs. 21(d), 22(d), and 23(d) demonstrate that increasing the number of bolts from #2 to #4 and #6 improves seismic performance by enhancing strength and stiffness. P_y increased from 125 kN (#2) to 178 kN (#4) and 185 kN (#6), and P_u from 278 kN to 394 kN and 411 kN, reflecting 41.9% and 47.9% gain in lateral resistance. K_e rose from 33 kN/mm to 38 kN/mm and 41 kN/mm, supporting better resistance to lateral deformation. Despite these gains, the deformation capacity beyond peak load decreased. DI declined by 15.9% from #2 to #4 and 20.4% from #2 to #6, as added bolts constrained post-ultimate plasticity. μ also dropped slightly (−7.8% and −8.2%), while Δ_u increased by 19.6% and 15.9% from #2 to #4 and #6, reflecting improved load redistribution and interface engagement. These results suggest that while more bolts boost strength and stiffness, they limit plastic deformation capacity, highlighting a trade-off between rigidity and ductility.

5.2.2 Effect of Diameter of Bolts

As shown in **Figs. 21(e), 22(e), and 23(e)**, increasing bolt diameter (22 mm → 30 mm → 42 mm) improves overall seismic performance, with P_y and P_u rising from 161 kN (357 kN) to 164 kN (365 kN) and 181 kN (401 kN), rising from 2.3% to 12.5%. K_e improved by 31% from 35 to 46 kN/mm, indicating enhanced shear resistance from larger bolts. DI increased modestly—by 9.1% (22→30 mm) and 3.8% (22→42 mm)—supporting better residual strength. However, the gains in

μ were inconsistent. μ rose slightly (2.8%) with 30 mm bolts but fell (2.6%) with 42 mm bolts. Similarly, Δ_u decreased as diameter increased, with a 16–19.6% reduction compared to 22 mm. This behavior suggests that larger bolts improve initial stiffness and strength but reduce deformation flexibility, especially in the plastic range. The stiffer interfaces limit displacement before failure, indicating that oversized bolts may reduce energy dissipation capacity in high-deformation zones.

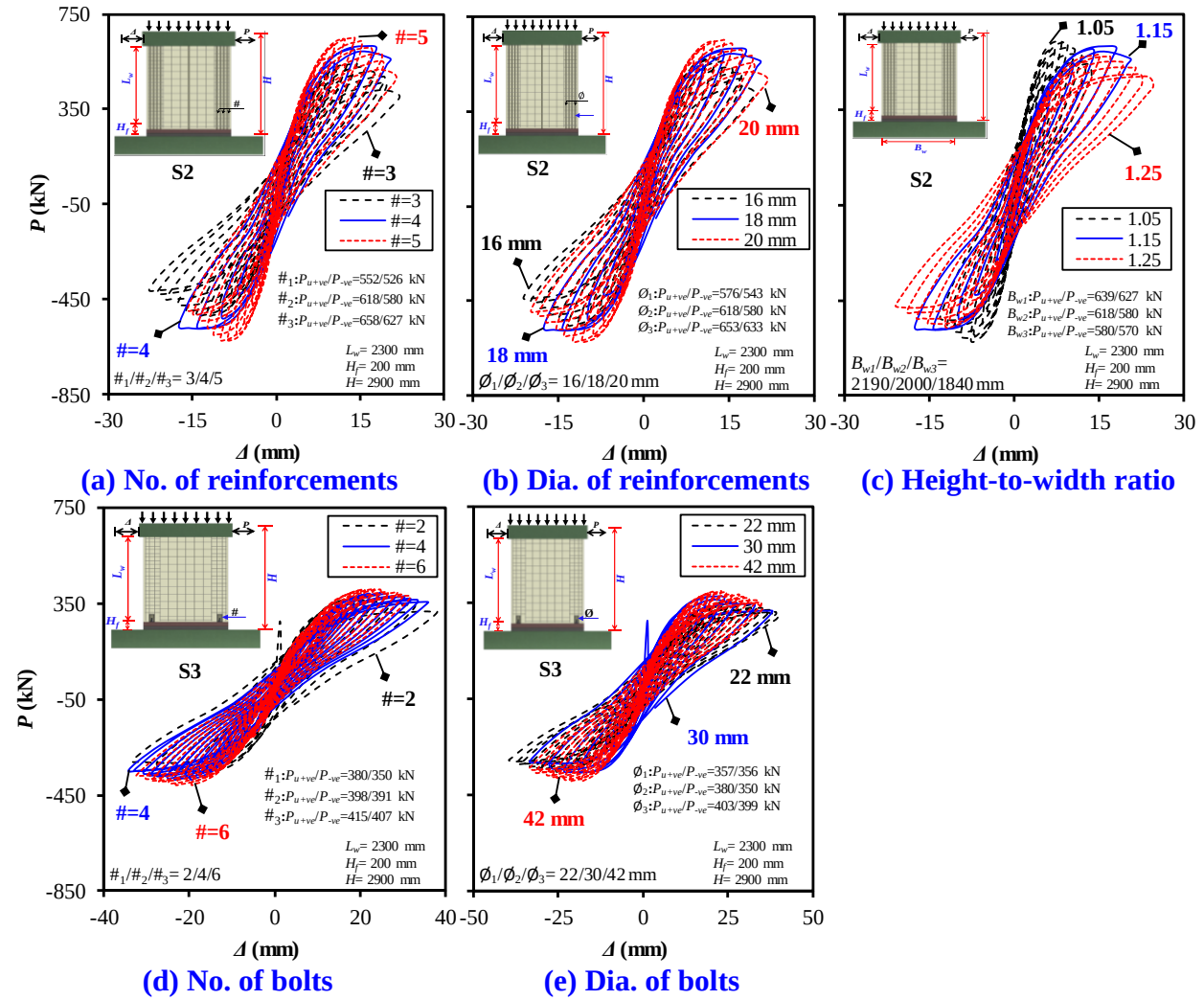


Fig. 21 Hysteresis curves from the parametric analysis of PVMS walls

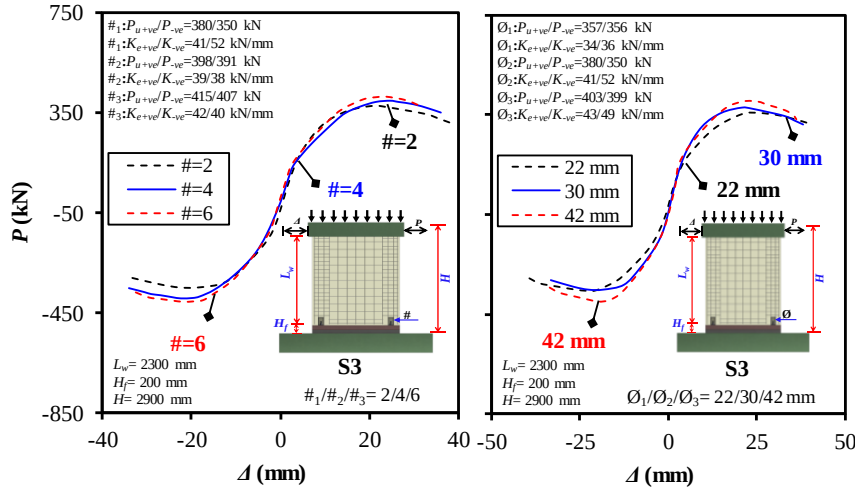
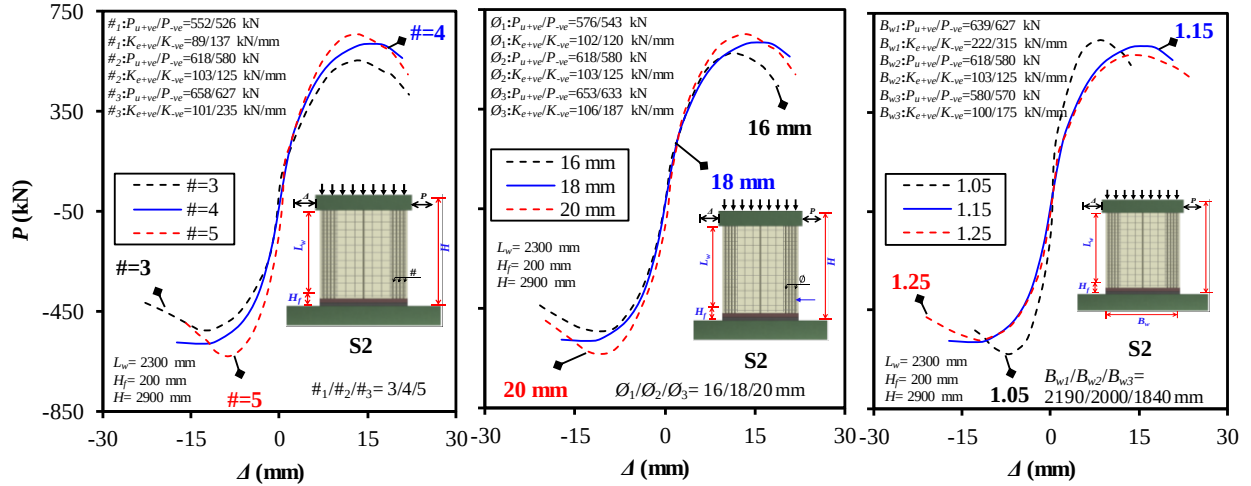
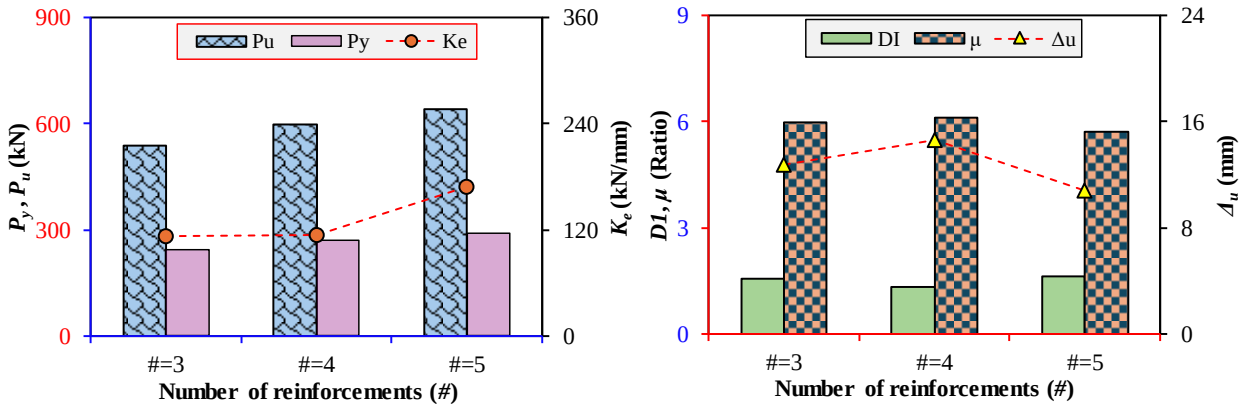
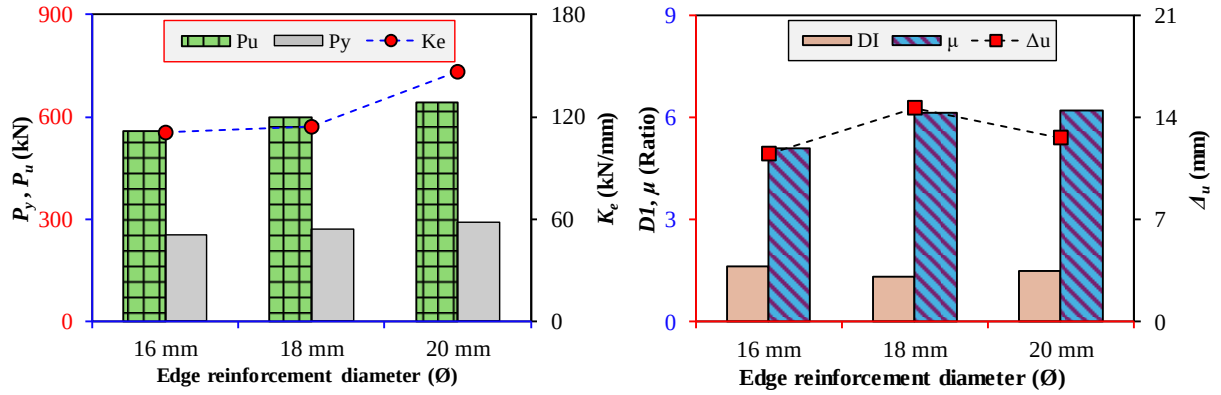
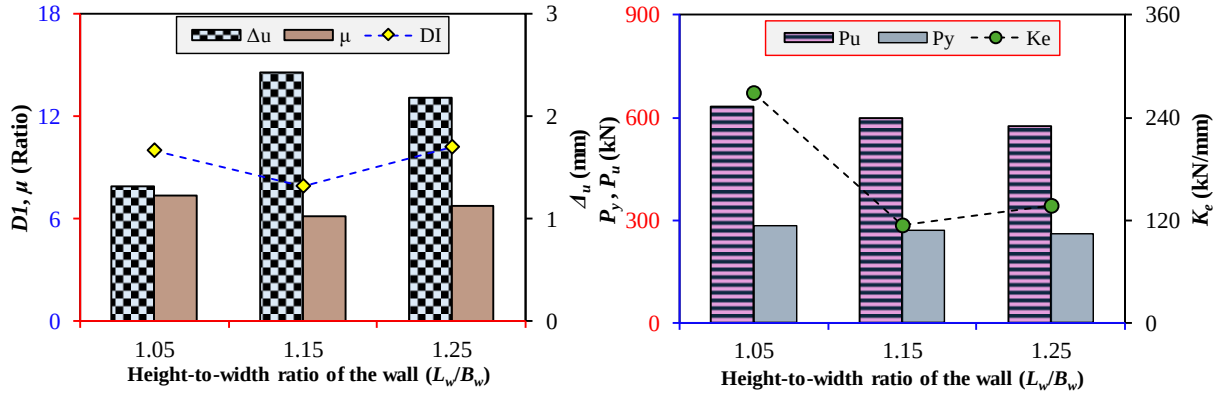


Fig. 22 Skeleton curves from the parametric analysis of PVMS walls

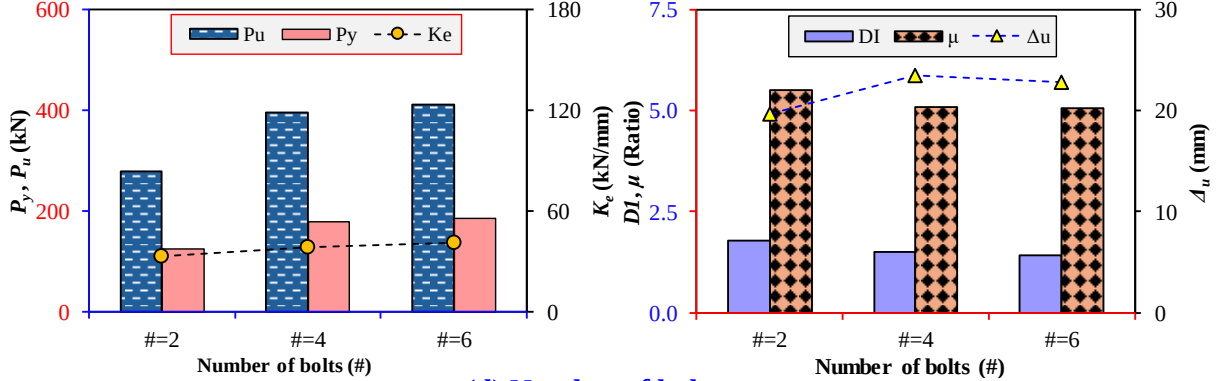




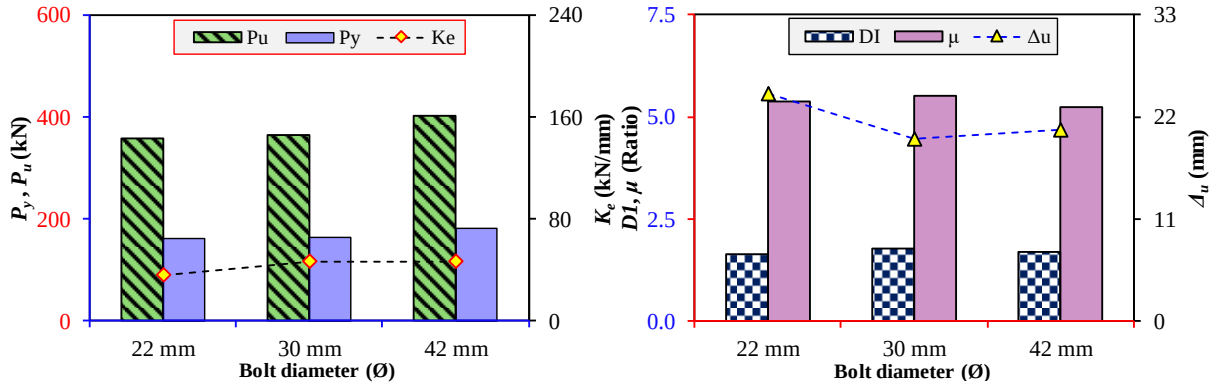
(b) Diameter of reinforcements



(c) Height-to-width ratio



(d) Number of bolts



(e) Diameter of bolts

Fig. 23 Impact of varying parameters on P_y and P_u to K_e and DI and μ to Δu curves in PVMS wall

6 Theoretical Study on the Flexural Load-Carrying Capacity of PVMS walls

Fig. 24(a-c) illustrates the simplified calculation model used to estimate the flexural load-carrying capacity of RC walls based on the JGJ 3–2010 [57]. These figures depict the stress distribution at the critical section of the wall. To facilitate analysis and develop a practical evaluation method, the following assumptions were adopted: (i) the concrete compression zone follows a rectangular stress block at ultimate capacity; (ii) the wall section remains plane before and after deformation (plane section assumption); (iii) the tensile strength of concrete is neglected; (iv) concrete in the compression zone attains full compressive strength; (v) out-of-plane behavior of the wall is disregarded; and (vi) longitudinal reinforcements in boundary elements are assumed to yield at failure.

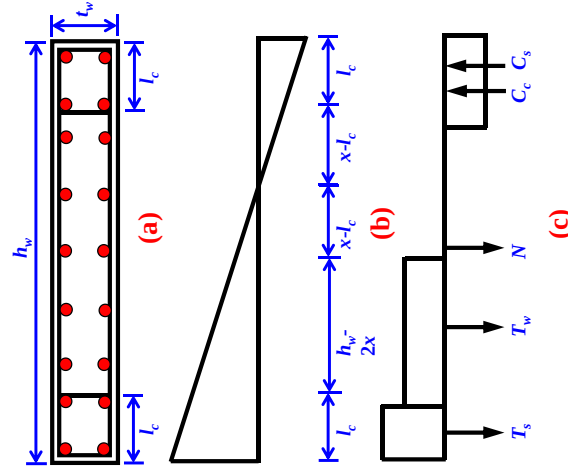


Fig. 24 Calculation diagram for PVMS walls load-carrying capacity

The force equilibrium of the section is expressed using **Eqn. 4** [57].

$$T_s + T_w + N = C_c + C_s \quad (4)$$

where T_s and C_s represent the tensile and compressive forces in steel reinforcements, respectively;

T_w is the vertical distribution reinforcement force; and C_c is the concrete compression force.

$$T_s = A_s f_y \quad (5)$$

$$C_s = A_s \sigma_s \quad (6)$$

$$C_c = 0.8 f_{ck} t_w x \quad (7)$$

$$T_w = (h_w - a_s - 1.5x) t_w f_y \rho_w \quad (8)$$

where h_w and t_w are the height and thickness of the wall section; l_s is the length of the edge column wall; x is the depth of the compression zone of the wall section; A_s and $\dot{A}_s$ denote the total cross-sectional areas of the longitudinal reinforcement in the tension and compression zones, respectively; σ_s is the average stress of the longitudinal reinforcement in the tension zone, calculated as $\sigma_s = E_s \frac{0.0033}{x} (x - a_s)$, a_s is the distance from the centroid of tensile reinforcement to the wall edge; ρ_w denotes the vertical reinforcement ratio; f_{ck} is the concrete compressive strength; and f_y is the yield strength of the steel bar.

Substituting **Eqns. 5-8** into **Eqn. 4** yields:

$$A_s f_y + (h_w - a_s - 1.5x) t_w f_y \rho_w + N = 0.8 f_{ck} t_w x + \dot{A}_s E_s \frac{0.0033}{x} (x - a_s) \quad (9)$$

Simplifying **Eqn. 9** to **Eqns. 10** and **11**:

$$x A_s f_y + x (h_w - a_s - 1.5x) t_w f_y \rho_w + x N = x (0.8 f_{ck} t_w x) + x [\dot{A}_s E_s 0.0033 (x - a_s)] \quad (10)$$

$$(0.8 f_{ck} t_w + 1.5 t_w f_y \rho_w) x^2 - (A_s f_y + (h_w - a_s) t_w f_y \rho_w + N - \dot{A}_s E_s 0.0033) x - \dot{A}_s E_s 0.0033 a_s \quad (11)$$

The compression zone depth, x , is calculated using **Eqn. 12**.

$$x = \frac{-B + \sqrt{B^2 - 4AC}}{2A} \quad (12)$$

where $A = 0.8 f_{ck} t_w + 1.5 t_w f_y \rho_w$, $B = A_s f_y + (h_w - a_s) t_w f_y \rho_w + N - \dot{A}_s E_s 0.0033$, and $C = -\dot{A}_s E_s 0.0033 a_s$

The maximum bending moment (M_1) is computed by **Eqn. 13**:

$$M_1 = T_s \left(\frac{h_w}{2} - a_s \right) + M_{sw} + C_c \left(\frac{h_w - 0.8x}{2} \right) + C_s \left(\frac{h_w}{2} - a_s \right) \quad (13)$$

Where:

$$M_{sw} = \frac{1}{2} (h_w - a_s - 1.5x) b_w f_y \rho_w (1.5x - a_s) \quad (14)$$

Finally, the theoretical lateral load-carrying capacity of PVMS is obtained using **Eqn. 15**.

$$P_t = \frac{M_1}{h_w} \quad (15)$$

6.1 Validation of Theoretical Model

The computed theoretical flexural capacities of S1, S2, and S3—calculated using **Eqn. 15**—were compared against experimental results averaged across both loading directions, as summarized in

Table 4. For S1 and S2, the theoretical-to-experimental ratios were 0.98 and 0.96, exhibiting 2.2% and 3.9% errors. These minor discrepancies validate the model's accuracy for monolithic and through-reinforced grouted wall assemblies, where the wall panel governs deformation, and the plane section assumption holds. However, for S3—a wall using fully bolted IMC—the ratio dropped to 0.65, underestimating the actual capacity by ~35%. This divergence stems from the inapplicability of the plane section assumption in bolted assemblies, where deformation localizes at the bolt-wall interface rather than being uniformly distributed across the wall section. Consequently, **Eqns. 11-15**, which are derived for integral behavior, do not fully capture S3's mechanical behavior. While the current theoretical model offers conservative predictions for bolted walls, it cannot account for unique load transfer mechanisms like bolt deformation, interface slip, and contact friction. Further refinements may involve empirical correction factors calibrated to experimental results or dedicated formulations incorporating interface friction, bolt pretension, and eccentric load paths to enhance predictive accuracy for such connections. Conclusively, the theoretical model provides reliable estimates for integrally cast (S1) or grouted (S2) walls but needs to be cautiously applied—or suitably modified—for bolted configurations (S3). Nonetheless, it remains a practical tool for preliminary design and conservative assessment of flexural capacity in modular systems.

7 Conclusions

This study evaluated the seismic performance of PVMS through full-scale cyclic tests on three wall systems: a cast-in-place reference wall (S1), a wall with a through-reinforced grouted IMC (S2), and a wall with a bolted IMC (S3). Experimental, numerical, and theoretical analyses were conducted to investigate failure mechanisms, load-displacement behavior, and the applicability of design models. The following conclusions were drawn:

1. S1 and S2 exhibited bending-dominated failure with diagonal cracking and concrete

spalling concentrated at the base corners near the floor-wall junctions. S3 demonstrated distinct sliding at the wall-floor interface, with localized damage near bolts but still governed by bending behavior overall.

2. S1 and S2 exhibited higher strength and stiffness. S2 slightly outperformed S1 in ultimate load (by 1.6%) but showed slightly lower ductility (DI : 1.10 vs. 1.33). In contrast, S3 exhibited 36.2% (65.7%) lower capacity (stiffness) than S1 but achieved the highest ductility (μ : 3.80) and energy dissipation (39,748.86 J), highlighting the trade-off between stiffness and deformability in bolted modular systems.

3. Finite element simulations using the CDP model aligned well with experimental data, with average prediction errors of 4.9% (P_u), 3.3% (Δ_u), and 10.6% (DI). Parametric analysis showed that increasing end reinforcement bars from #3 to #5 improved capacity by 19.3% and stiffness by 48.7% while increasing bar diameter from 16 mm to 20 mm improved capacity by 14.9% and stiffness by 31.9%. Increasing the height-to-width ratio from 1.05 to 1.25 resulted in reductions of 9.1% in capacity and 48.9% in stiffness. Increasing bolt quantity from #2 to #6 improved capacity (stiffness) by 47.9% (23.3%), while increasing bolt diameter from 22 mm to 42 mm enhanced capacity by 12.5% and stiffness by 30.7%.

4. The flexural capacity model based on JGJ 3–2010 accurately predicted the capacities of S1 and S2, with minor errors of 2.2% and 3.9%, confirming its suitability for integrally cast or grouted walls. However, it underestimated S3's capacity by 35% due to localized bolt deformation and slip, which violate the plane section assumption. This limitation indicates the need for model adjustments or correction factors when applied to bolted assemblies.

This study confirms the viability of vertical IMCs—through-reinforced grouted and bolted—for multi-story PVMS applications. S2 demonstrates strength and durability suitable for high-seismic

regions, while S3 enables rapid modular deployment in lower-risk scenarios. Future research will focus on horizontal IMCs between floor and ceiling slabs to enhance module integration, aiming to improve both vertical and lateral continuity in resilient high-rise PVMS designs.

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