

Article

Seismic Performance of Full-Scale Autoclaved Aerated Concrete Panel-Assembled Walls: Experimental Study and Numerical Modeling

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Abstract: Prefabricated panel-assembled wall systems, comprising a confining frame and infill lightweight panels of autoclaved aerated concrete (AAC), are widely employed in framed structures. Different from studies on a main frame with infill walls, this study aimed to explore the seismic performance of partition walls, which were fabricated with AAC panel-assembled walls and located outside of the main frames. Two full-scale specimens, one with a door opening and the other without, were constructed and cyclic loading tests were executed to examine the failure modes, hysteresis characteristics, envelope curves, ductility, strength and stiffness degradation, as well as energy dissipation capacity of the AAC panel-assembled walls. Additionally, a restoring-force model for the panel-assembled walls was developed and a method for predicting the lateral load-bearing capacity of the AAC panel-assembled walls was proposed. The findings indicated that the panels enhanced the system's lateral resistance, energy dissipation capacity, and deformation capability. The door frame increased the initial stiffness, peak lateral load and energy dissipation capacity of the AAC panel-assembled wall compared to the wall without a door frame. Compared to the specimen without a door frame, the peak lateral load of the specimen with a door frame increased by 19.7–30.1%. The deformation capacity of the panel-assembled walls aligned with the requirements for concrete framed structures.

Keywords: AAC panels; infill walls; prefabricated members; confining frame; tie column



Citation: Li, X.; Ma, D.; Zhang, Q.; Zhang, Z.; Bao, H.; Yao, Y. Seismic Performance of Full-Scale Autoclaved Aerated Concrete Panel-Assembled Walls: Experimental Study and Numerical Modeling. *Buildings* **2024**, *14*, 1333. <https://doi.org/10.3390/buildings14051333>

Academic Editor: Pejman Sharafi

Received: 7 April 2024

Revised: 30 April 2024

Accepted: 3 May 2024

Published: 8 May 2024



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1. Introduction

Traditionally, infill walls are constructed using bricks or concrete blocks. Nevertheless, with the construction industry increasingly embracing sustainability and witnessing a rise in prefabricated buildings, materials for infill walls are progressively shifting toward autoclaved aerated concrete (AAC) blocks and panels. Due to its merits of light weight and thermal and sound insulation efficiency, AAC has been widely used in the interior and exterior walls of buildings.

Infill walls have been commonly employed to segment the internal space of buildings into individual rooms. Research has shown that infill walls between frames not only influence the dynamic characteristics, lateral resistance, and lateral stiffness of frame structures but also undergo damage during seismic events [1].

There are two kinds of seismic design philosophies for infilled frames in view of the diagonal strut effect of infill walls: one is improvement of the overall integrity and strength of infill walls, and the other is the isolation of infill walls from structural systems through gaps set between the infill walls and the main frames [2,3]. In recent years, numerous researchers have explored the seismic performance of framed structures with AAC infill walls. Sucuoğlu et al. (2014) conducted pseudo-dynamic testing and presented an analytical modeling of AAC infilled RC frames, drawing a conclusion that AAC infill panels do not

modify the deformation response of the RC frame, and the diagonal strut effect of AAC infill panels must be considered in confining column design [4]. Schwarz et al. (2015) explored the influence of AAC masonry infill walls on concrete frames, and pointed out that as compared with a frame without infill walls, a frame with masonry infill walls has a larger lateral load bearing capacity and lower ductility, and that openings in masonry infill walls produce capacity and ductility values that are intermediate between those of the frame with full infill and the frame without infill [5]. Wang et al. (2017) studied frames with autoclaved lightweight concrete (ALC) walls and indicated that embedded ALC walls can improve the rigidity and strength of frames, and that a frame with ALC walls displays good cooperative behavior and safety reliability [6]. Xu et al. (2021) analyzed the seismic behavior of AAC prefabricated panel walls and indicated that AAC panel-walls have higher seismic shear capacity as compared with block walls, and that using constructional columns can enhance the ultimate load-bearing capacity of the wall [7].

In order to limit infill frame interaction, Jin et al. (2021) presented a new type of wall configuration to isolate infill walls from the surrounding RC frames, conducted cyclic tests on RC frames with isolated infill, and found that frame-isolated infill can effectively reduce infill wall damage and undesirable infill–frame interactions [8]. Ding et al. (2022) carried out a cyclic test of a steel frame with ALC panels, proposed a novel pendulous Z-plate connector, and indicated that the new connector can reduce the damage of ALC panels and maintain the safety of the steel frames [9]. Li et al. (2023) carried out experiments on steel frames with an infill ALC wall, and indicated that the infill ALC wall panels greatly enhance the seismic performance of the frames. A steel frame with infill wall panels has better ductility and energy-dissipating capacity than a bare frame, and that the panels delay the buckling and in-plane deformation of the frame due to cooperation with the steel frame [10]. Wang et al. (2024) studied the influence of embedded prefabricated ALC wall panels on the seismic performance of assembled partially encased concrete T-shaped column frames, and indicated that the stiffness contribution of infill walls is primarily affected by local compression, and that the presence of ALC walls can significantly increase the initial stiffness of the assembled frame [11].

Previous studies primarily concentrated on infill walls constructed from materials such as bricks, concrete blocks, AAC blocks, and AAC panels. Bricks, concrete blocks, and AAC blocks were affixed to the columns of framed structures using horizontal reinforcement, while panels were attached through connectors such as T-shaped or U-shaped connectors, or L-shaped bolts. These studies typically focused on scaled frame specimens with infill walls, without considering the size effects.

This study aimed to explore the seismic performance of independent partition walls, which are fabricated with AAC panel-assembled walls and located outside of the main frames. A fabrication method for panel-assembled walls was introduced, and two full-scale specimens, one with a door opening and one without, were assembled. Cyclic loading tests were then conducted to study the seismic performance of the AAC panel-assembled walls. Furthermore, the stress distribution characteristics of the AAC panel-assembled walls were examined using the finite element (FE) method, a restoring-force model for such walls was subsequently proposed, and a method for predicting the lateral load-bearing capacity of AAC panel-assembled walls was proposed.

This study consists of nine sections. The first section introduces the research background and objectives for the AAC panel-assembled partition walls. Sections 2–5 cover the experiment preparation and results, encompassing material properties, test setup, instrument arrangement, loading procedure, and results and analyses. In the “Results and Analyses” section, detailed experimental results are provided for both AAC panel-assembled walls with and without door openings, focusing on failure phenomena, hysteresis curves, envelope curves, ductility index, inter-story drift rotation, strength degradation, stiffness degradation, energy dissipation, and shear angle. In Section 6, a recovery force model is established, where the recovery force model is divided into two parts: the envelope model and the hysteresis rule. Section 7 discusses the material models used in FE analyses, bound-

any conditions, meshing, contact, connection, loading procedure, and stress distribution. It also includes a comparison of the results of the hysteresis curves and envelope curves obtained from the tests and FE analyses. In Section 8, a calculation model for the lateral load-bearing capacity of AAC panel-assembled walls is proposed, including methods to calculate the failure load of a single diagonal brace and the yielding resistance of a confining frame. Section 9 summarizes the entire study and provides the conclusions. The research framework is illustrated in Figure 1.

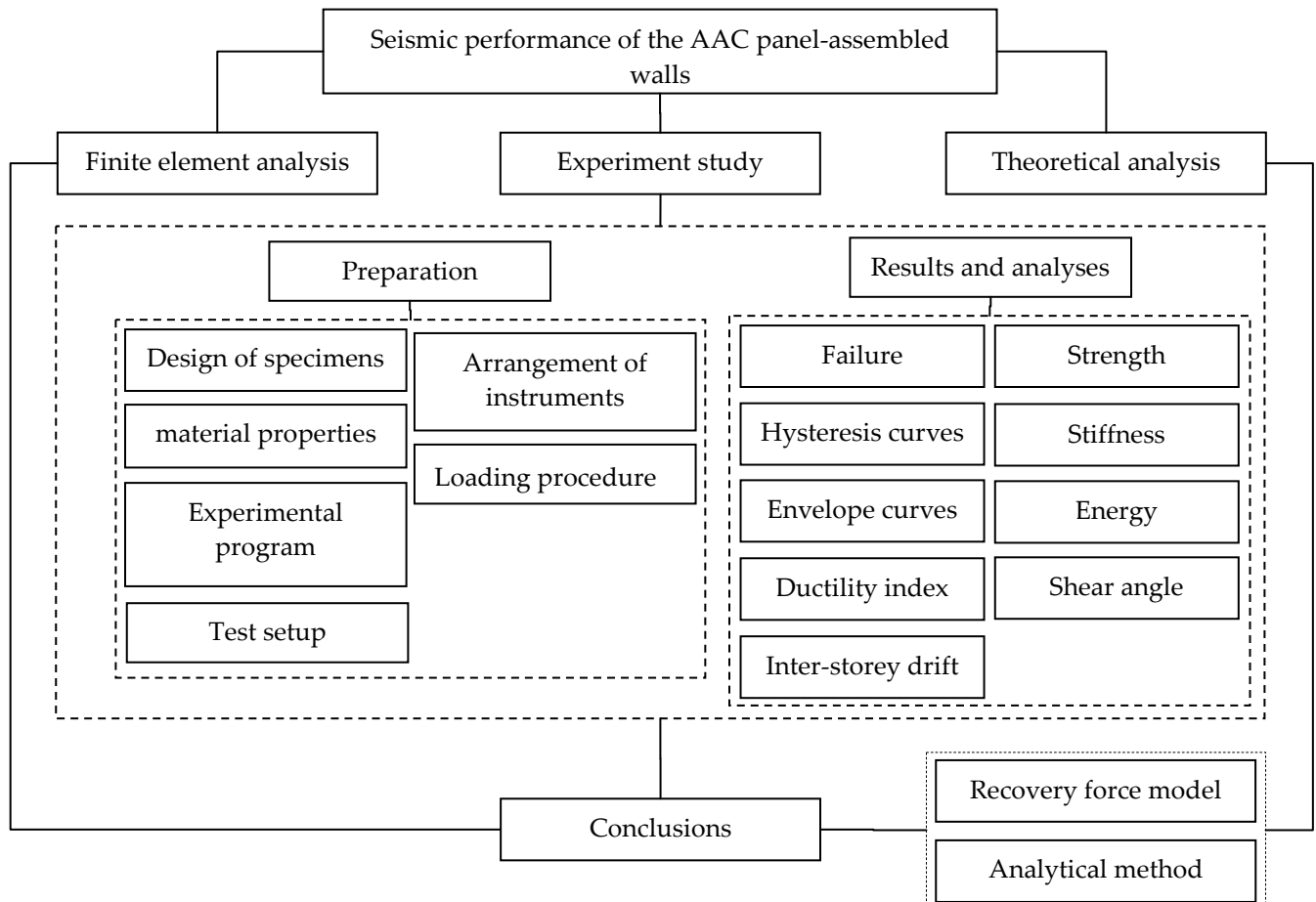


Figure 1. Research framework.

2. Design of Specimens

In large-span framed structures, due to the need for spatial division, walls are not necessarily placed entirely within the frames; instead, lightweight partition walls are often directly installed on the floor outside of the frames. As depicted in Figure 2, the partition wall ‘P’ is not directly connected to the structural columns and beams. According to the Chinese standard of Code for Seismic Design of Buildings (GB 50011-2010) [12], tie columns need to be appropriately placed in the partition wall, with a spacing not exceeding 4 m. To enhance the entirety of the partition wall, connecting beams are intentionally positioned at the top and bottom of the wall. Through this arrangement, nonstructural columns and beams can collaboratively form a confining frame, strengthening the partition wall. Additionally, the nonstructural beam located at the base of the partition wall serves to impede the transmission of floor moisture along the AAC panels.

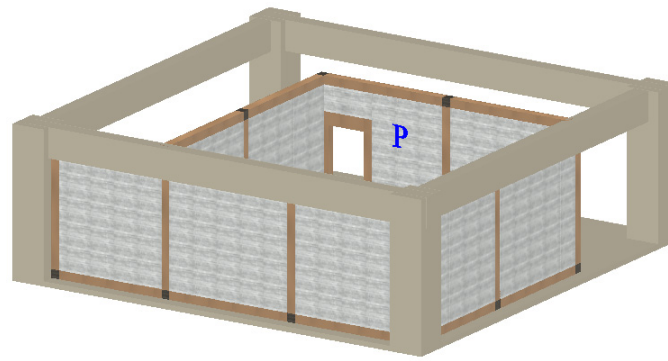


Figure 2. Partition walls in framed buildings.

According to Figure 2, two AAC panel-assembled walls were designed. Specimen W1, presented in Figure 3a, measured 3.4 m in width and height, with an opening of 1 m $\times$ 2.3 m (width $\times$ height). W1 included a confining frame, a door frame, three vertical panels and one horizontal panel. The confining frame was made of two prefabricated columns (labeled as C1) and two prefabricated beams (labeled as B1 and B2). The door frame was made of two door columns (labeled as C2 and C3) and a door beam (labeled as B3). The width of the two vertical panels and the horizontal panels was 0.6 m (labeled as P1 or P3), and the wide of one vertical panel was 0.5 m (labeled as P2). In Figure 3b, specimen W2, also measuring 3.4 m in width and height, incorporated a confining frame and five vertical panels. The confining frame of W2 consisted of two prefabricated columns (labeled as C1) and two prefabricated beams (labeled as B1). The panels of W2 were arranged vertically, and the width of the panels was 0.6 m (labeled as P1).

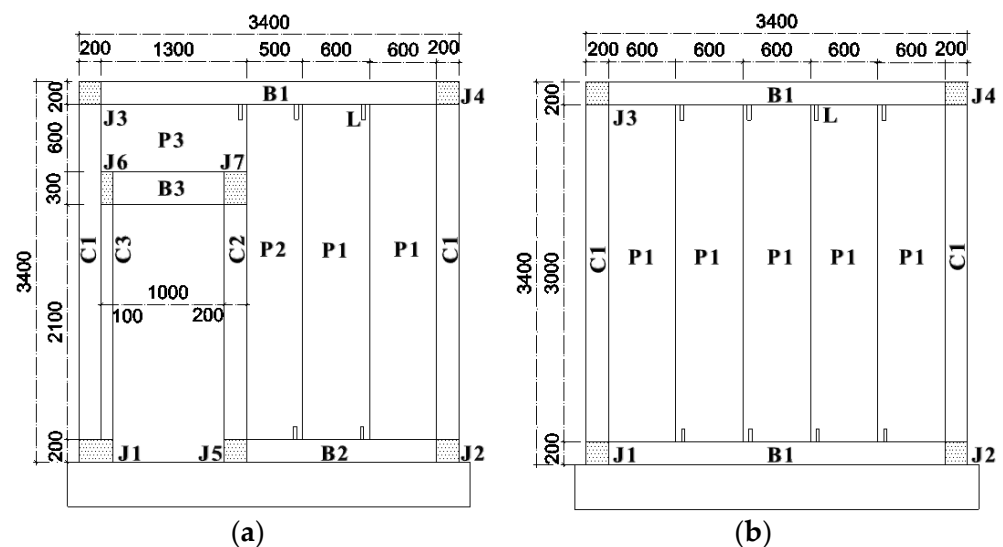


Figure 3. Specimens. (a) W1, (b) W2 (in mm).

In Figure 3, the panels and the prefabricated components were connected through mortise and tenon and L-shaped connectors (represented as 'L' in the Figure 3). The L-shaped connectors were constructed by welding a 1.5 mm thick steel pipe with a diameter of 15 mm and a 3 mm thick steel plate, as depicted in Figure 4a. Prefabricated components were connected by cast-in-place concrete (as shown in Figure 4b) and represented as 'J' in Figure 3. After the prefabricated components were installed, stirrups were welded onto the reserved longitudinal steel bars at the ends of the components. Subsequently, molding boards were installed, and concrete was poured in the joint regions.

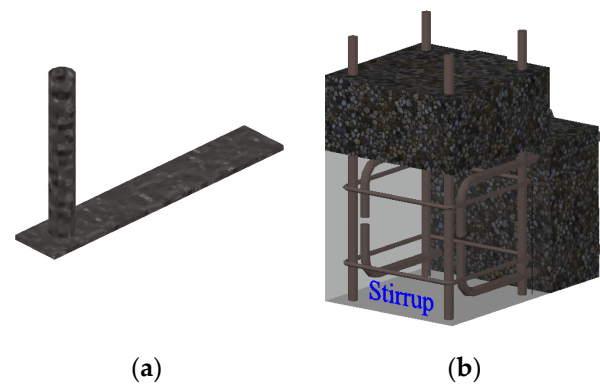


Figure 4. Configurations. (a) L-shaped connector, (b) details of cast-in-place joints.

The reinforcement and cross-sectional dimensions of the AAC panels are depicted in Figure 5a. The cross-sections and reinforcement details of the prefabricated components can be found in Figure 5b–d. The dimensions of the prefabricated components and AAC panels are provided in Table 1.

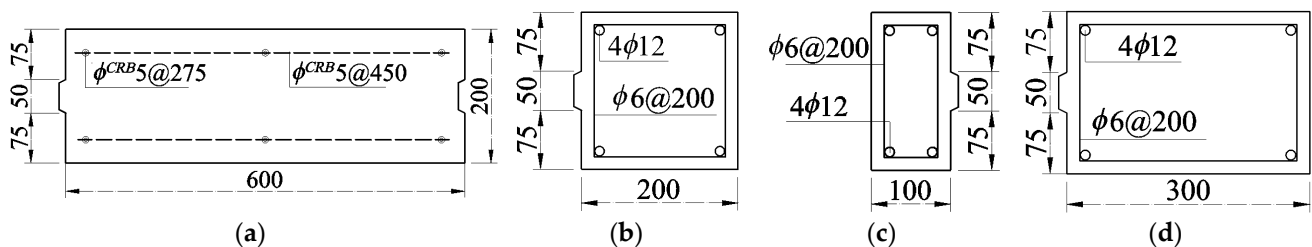


Figure 5. Sizes and reinforcement of components. (a) P1; (b) C1, C2, B1 and B2; (c) C3; (d) B3 (in mm).

Table 1. Sizes of prefabrication components.

Names	Labels	Width mm	Depth mm	Length mm
Columns	C1	200	200	3000
	C2	200	200	2100
	C3	100	200	2100
	B1	200	200	3000
Beams	B2	200	200	1700
	B3	200	300	1300
Panels	P1	600		3000
	P2	500	200	3000
	P3	600		1300

In the prefabricated components, the stirrup had a diameter of 6 mm and a grade of HPB300, while the longitudinal reinforcement had a diameter of 12 mm and a grade of HRB400. The AAC panels had a density of 0.8 kN/m^3 and a designed compressive strength of 5.0 N/mm^2 . The panels were internally reinforced with a double layer of welded web, featuring 5 mm diameter steel rod.

The installation sequence for the AAC panel-assembled wall was as follows:

- Install the prefabricated components;
- Pour cast-in-place joints;
- Install panels after the concrete in the joints reaches the design strength;
- Hammer the steel pipe of the L-shaped connector into the end of the panels;
- Align the panel with the confining frame;
- Move the panel into the intended position;
- Fasten the L-shaped connectors to the confining beam using explosive pins.

3. Material Properties

The prefabricated components employed commercial concrete with a design strength of C30 (Mianyang Chielong Concrete Co., LTD., Mianyang, China). Concrete with grade of C30 was utilized for the cast-in-place joints and the mix ratios of the cast-in-place concrete are outlined in Table 2.

Table 2. Mix ratios of concrete for joints in 1 m³.

Cement	Coarse Aggregates	Sand	Fly Ash	Water Reducer	Water
kg	kg	kg	kg	kg	kg
371	901	752	250	22	153

The designated concrete grade for both the prefabricated components and joints was C30. In accordance with the literature [13], the parameters of the mechanical performance for the concrete were tested. The standard compressive strength for the concrete cubes with sizes of 150 mm × 150 mm × 150 mm was 32.41 N/mm² for both the prefabricated components and cast-in-place joints. Additional mechanical parameters of the concrete were derived from the Chinese standard of the Code for design of concrete structures (GB 50010-2010) [14] and are given in Table 3. The standard compressive strength for the AAC cubes was 5.17 N/mm².

Table 3. Mechanical parameters of concrete and steel bars.

		Concrete			Steel				
		PC ^a	Joints	AAC			HPB300	HRB400	CRB550
$f_{cu,ave}$ ^{b1}	N/mm ²	33.00	34.63	5.30	Diameter	mm	6	12	5
C.V. ^c		0.011	0.039	0.015					
$f_{cu,k}$ ^{b2}	N/mm ²	32.41	32.41	5.17	f_y ^{f1}	N/mm ²	410	407	500
f_{ck} ^{b3}	N/mm ²	21.68	21.68	--					
f_{tk} ^{d1}	N/mm ²	2.10	2.06	--					
f_c ^{b4}	N/mm ²	15.48	15.48	--	f_u ^{f2}	N/mm ²	511	592	550
f_t ^{d2}	N/mm ²	1.50	1.47	0.5					
E_c ^{e1}	N/mm ²	30,576	30,576	2100	E ^{e2}	10 ³ × N/mm ²	193	205	200

^a Precast concrete; ^{b1} Average compressive strength of concrete cubes; ^{b2} Standard compressive strength for the concrete cubes; ^{b3} Standard compressive strength for concrete prisms; ^{b4} Design compressive strength for concrete prisms; ^c Coefficient of variation; ^{d1} Standard tensile strength for concrete; ^{d2} Design tensile strength for concrete; ^{e1} Elastic modulus of concrete; ^{e2} Elastic modulus of steel reinforcements; ^{f1} Yield tensile strength of reinforcements; ^{f2} Maximum tensile strength of reinforcements.

The diameter of the stirrups was 6 mm, with a grade of HPB300. The diameter of longitudinal reinforcements was 12 mm, with a grade of HRB400. And the diameter of cold-rolled ribbed steel bars used in AAC panels was 5 mm, with a grade of CRB 550. According to the Chinese standard of Cold Rolled Ribbed Steel Wires and Bars (GB/T 123788-2017) [15], the yield tensile strength of CRB 550 was 500 N/mm². The mechanical parameters of the reinforcement bars are given in Table 3.

4. Experimental Program

4.1. Test Setup

The specimens were affixed to the base through embedded steel plates, and the horizontal loading equipment used was an electro-hydraulic servo loading system manufactured by the corporation of Mechanical Testing & Simulation, Eden Prairie, MN, USA. To prevent out-of-plane displacement during the experiment, two lateral support beams were strategically installed on both the front and back sides at the top of the specimens, as depicted in Figure 6.

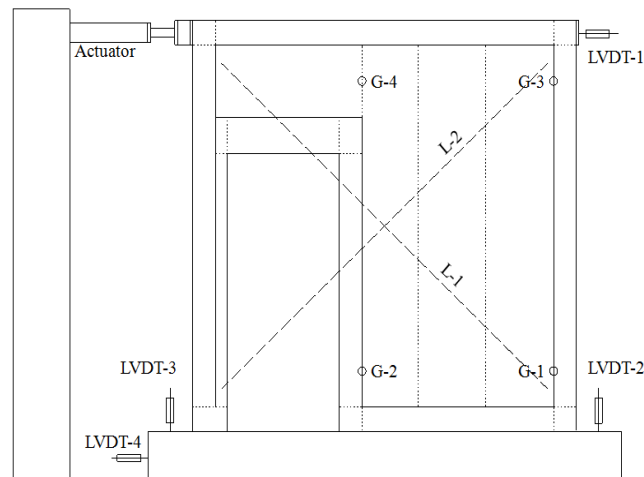


Figure 6. Setup and arrangement of instruments.

4.2. Arrangement of Instruments

The configuration of the instruments and gauges is depicted in Figure 6. To capture the true drift of the specimens, LVDT-1 was positioned on the top right of the specimen, while LVDT-4 was placed at the bottom. LVDT-2 and LVDT-3 were strategically located on both sides of the foundation's top. To measure the shear deformation in the specimens, two diagonal wire displacement gauges, L-1 and L-2, were installed. Four displacement gauges, namely G-1, G-2, G-3, and G-4, were arranged to assess the relative sliding between the wall panels and the prefabricated columns. To detect the concrete strain, five 100 mm-long strain gauges were positioned along the surface of the prefabricated components at each section 100 mm from every joint. Furthermore, for monitoring the rebar strain, 10 mm-long strain gauges were applied to longitudinal bars located 100 mm from each joint.

4.3. Loading Procedure

In accordance with literature [16–20], the predetermined loading procedure is depicted in Figure 7. The specimen's rightward movement was considered positive, while leftward movement was negative, corresponding to the actuator's push and pull directions. The loading method was displacement-controlled, where each level cycled once at intended drifts of 10 mm, 20 mm, and 30 mm. At the intended drifts range from 40 mm and 120 mm, each level cycled thrice, increasing the intended drift by 20 mm. Starting from 120 mm, each level repeated thrice, incrementing the intended drift by 30 mm per level. The cycling continued until the lateral load decreased to 85% of the maximum load, at which point the test stopped.

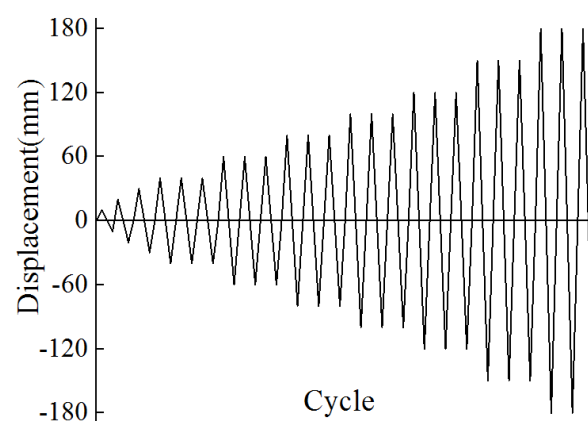


Figure 7. Loading procedure.

4.4. Results and Analyses

4.4.1. Failure Phenomenon

During the experimental process, as the drift increased, cracks initially appeared on the prefabricated column C1, followed by the occurrence of cracks on the upper prefabricated beam B1. When the drift became significant, inclined cracks appeared at the cast-in-place joints and localized shear failure occurred at the corner of the AAC panels and at the cast-in-place joints. The ultimate final failure modes are illustrated in Figure 8.

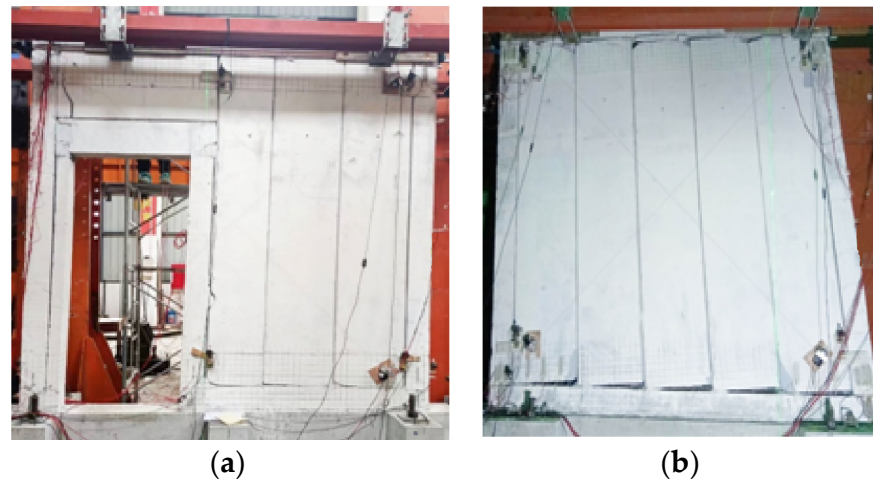


Figure 8. Failure of specimens. (a) W1, (b) W2.

When the drift was less than 20 mm, the lateral load was borne by the confining frame due to a 20 mm installation gap on both sides of the wall panels. The interaction force between the AAC panels and the confining frame was relatively small, but due to the presence of L-shaped connectors and mortise-tenon joints, the panels moved along with the confining frame. As drift increased, the interaction between the panels and the frame gradually strengthened, primarily manifested in the upward bending of the top beam of the confining frame due to the uplift of the corners of the panels. At a drift of 10 mm, horizontal flexural cracks initiated in the prefabricated column C1, at heights ranging from 0.85 m to 1.12 m (see Figure 9a).

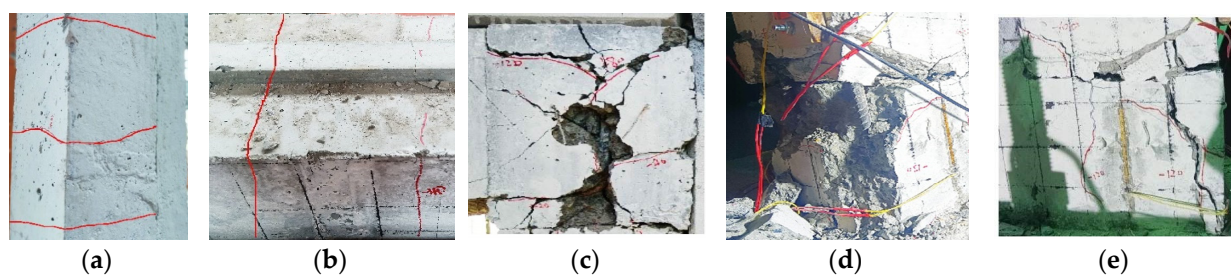


Figure 9. Cracks. (a) Cracks on the column, (b) cracks on the beam, (c) cracks on the upper right joint of the door frame, (d) cracks on the lower left joint, (e) cracks on the lower right joint of the confining frame.

In the drift range of 20–80 mm, under cyclic loading, the lateral load was mainly borne by the confining frame and shared by the panels through their interaction. When the drift reached 20–30 mm, the longitudinal steel bar yielded, and vertical flexural cracks emerged in the prefabricated beam B1 due to the uplift rotation of the AAC panels (see Figure 9b). In the range of 50 mm to 60 mm drift, inclined cracks initiated at the upper joints of the door frame and the upper joints of the confining frame. At 80 mm drift, the lateral force reached its maximum, leading to concrete spalling at the joints of the door frame (see Figure 9c).

When the drift was larger than 80 mm, severe damage occurred at the corners of the confining frame, forming ‘joint hinges’ (see Figure 9d,e). The lateral load borne by the confining frame gradually decreased. Due to the inclination and uplift of the AAC panels, the diagram strut effect of the AAC panels on the confining frame strengthened, and the lateral load was primarily borne by the panels. This increased support enhanced the ductility, energy dissipation, and deformation capability of the panel-assembled wall system.

With increasing drift, the corners of the AAC panels rotated and uplifted, resulting in shear failure at the corners of the panels (see Figure 10a,b).

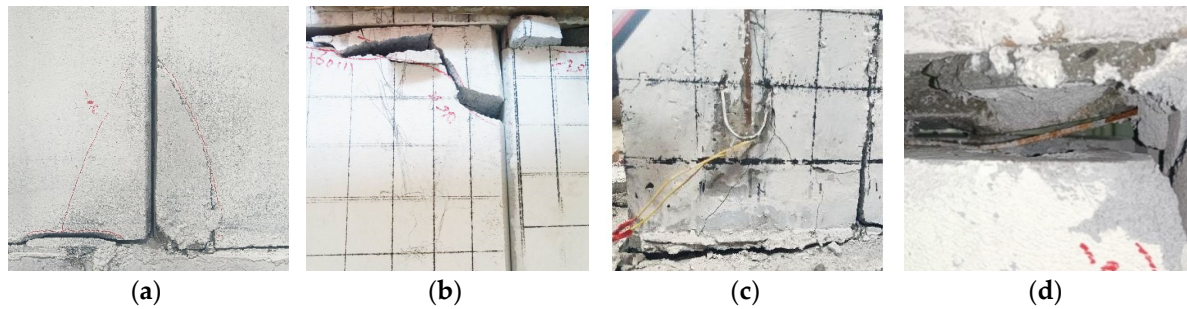


Figure 10. Failure. (a) Cracks on the lower corners, (b) cracks on the upper corners of the panels, (c) failure of the weld seam, (d) deformation of the L-shaped connectors.

At a drift of 210 mm, the weld seam of the embedded steel plate in the left prefabricated column C1 of specimen W1 fractured (see Figure 10c). With increasing drift, the gap between the panels and the top beam of the confining frame became wider, and the L-shaped connectors between the panels and the top beam deformed with the rotation of the panels, as shown in the Figure 10d.

At a drift of 250 mm, a significant amount of concrete spalling occurred at the cast-in-place joints of the specimens, causing a rapid decrease in lateral load-bearing capacity. Throughout the experimental process, careful observation of crack development on the specimen’s surfaces was conducted. The maximum crack lengths and widths on the specimen’s surfaces at yield, peak, and ultimate states are detailed in Table 4. The rotational uplift of concrete panels was observed during the experimental process, and the uplift heights are also provided in Table 4.

Table 4. Cracks and uplift of AAC panels.

Specimens States	Yielding		W1 Peak		Ultimate		Yielding		W2 Peak		Ultimate	
	L ^a (mm)	W ^b (mm)	L (mm)	W (mm)	L (mm)	W (mm)	L (mm)	W (mm)	L (mm)	W (mm)	L (mm)	W (mm)
Prefabricated columns	200	0.55	200	1.75	200	3.2	200	0.65	200	1.5	200	3.3
Prefabricated beams	--	--	200	1.5	200	2.4	--	--	200	1.3	200	2.5
Joints	210	0.65	238	3.5	282	5.5	--	--	243	3.2	265	4.6
AAC panels	84	2.5	250	2.8	320	5.6	36	1.65	262	3.5	285	4.5
Uplift of panels	3.26		13.5		24.6		3.03		11.45		22.4	

^a Maximum length of the cracks; ^b Maximum width of the cracks.

4.4.2. Hysteresis Curves

The hysteresis curves are shown in Figure 11. When the drift was less than 30 mm, the hysteresis curves exhibited a ‘spindle’ shape, and at this moment, cracks primarily appeared on the prefabricated columns and beams, with minimal residual deformation after unloading. As the drift increased from 30 mm to 80 mm, the hysteresis curves gradually transformed into an ‘S’ shape, leading to the initiation of cracks in the joints of the confining

frame. When the drift exceeded 80 mm, the hysteresis curves gradually transformed into a 'Z' shape. With increasing drift, cracks in the cast-in-place joints gradually extended, with their width gradually increasing, forming a 'joint hinge.' In the range of 80 mm to 150 mm drift, the concrete in the joint region gradually peeled off, resulting in a gradual reduction in lateral load-bearing capacity. When the drift reached 100 mm to 120 mm, the lateral load-bearing capacity dropped to a minimum value. At this stage, the rotation angle of the AAC panels gradually increased, and the interaction between the AAC panels and the confining frame increased. With a drift of 120 mm to 150 mm, due to the enhanced interaction between the AAC panels and the confining frame, the lateral load-bearing capacity recovered from the minimum value. Continuing to increase the drift, the corners of AAC panels gradually underwent damage, and the interaction between the AAC panels and the confining frame began to decrease, leading to a gradual reduction in lateral load-bearing capacity.

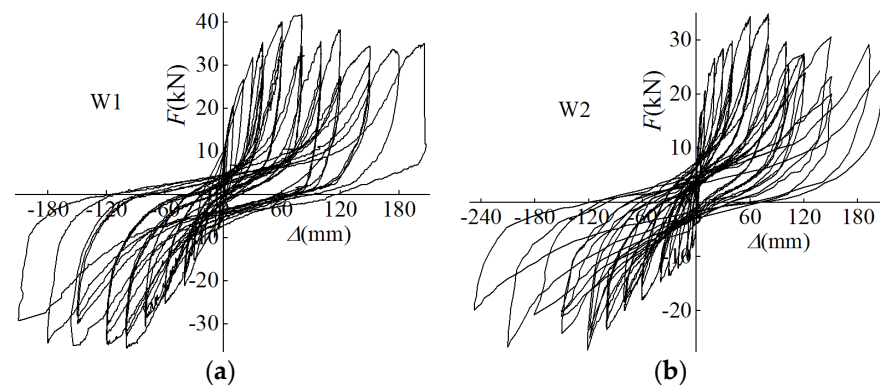


Figure 11. Hysteresis curves. (a) W1, (b) W2.

4.4.3. Envelope Curves

The geometric plotting method was used to determine the key points. The force–drift envelope curves are illustrated in Figure 12. From the graph, it is evident that the envelope curves for panel-assembled wall W1 and the specimen W2 exhibited similar shapes and development trends. The cracking loads and cracking drifts were close. Under the same intended drift, the positive lateral load of the envelope curves was greater than the corresponding negative. The panels played a crucial role in supporting the confining frame, thereby enhancing the lateral load-bearing capacity and the ductility of the panel-assembled walls.

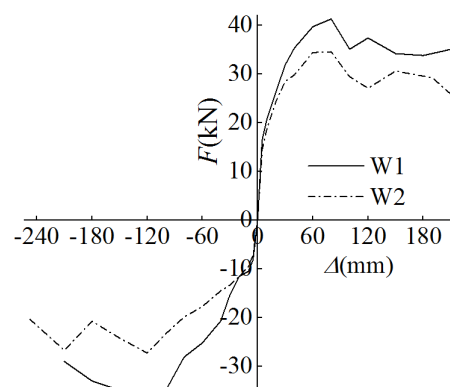


Figure 12. Envelope curves.

The interaction between panels and the confining frame led to a 'decreasing—increasing—decreasing' process in the descending segment of the envelope curves. In this process, 'joint hinges' formed, reducing the lateral load-bearing capacity of the confining frame. As drift continued to increase, the panels rotated and uplifted, increasing their support to the frame. After the shear failure of the corners of the panels, their supporting effect decreased.

Table 5 illustrates the key points of the envelope curves. The door frame could enhance the initial stiffness and peak load of the panel-assembled wall. Compared to specimen W2, in the positive direction, the yield load, peak load, and ultimate load of specimen W1 with a door frame increased by 18.9%, 19.7%, and 19.7%, respectively, while in the negative direction, the yield load, peak load, and ultimate load of specimen W1 increased by 9.8%, 30.1%, and 30.1%. The negative yield load of specimen W1 was about 0.48 times its positive yield load, whereas for specimen W2, it was 0.52 times its positive yield load.

Table 5. Key points.

Key Points	Symbol	W1		W2	
		Push (+)	Pull (−)	Push (+)	Pull (−)
Crack	Δ_{cr} ^{a1} mm	10.00	10.00	10.00	10.00
	F_{cr} ^{b1} kN	20.70	10.48	18.71	9.48
	θ_{cr} ^{c1} Rad	1/330	1/330	1/330	1/330
Yield	Δ_y ^{a2} mm	20.90	22.40	22.40	16.50
	F_y ^{b2} kN	26.40	12.63	22.20	11.50
	θ_y ^{c2} Rad	1/158	1/147	1/147	1/200
Peak	Δ_p ^{a3} mm	80.20	100.00	82.50	121.30
	F_p ^{b3} kN	41.40	35.40	34.60	27.20
	θ_p ^{c3} Rad	1/41	1/33	1/40	1/27
Ultimate	Δ_u ^{a4} mm	206.36	197.77	191.52	231.20
	F_u ^{b4} kN	35.20	30.09	29.41	23.12
	θ_u ^{c4} Rad	1/16	1/17	1/17	1/14
DI ^e	β	9.87	8.83	8.55	14.01
ADI ^f	β_{ave}	9.35		11.28	

^{a1} Cracking drift; ^{a2} Yield drift; ^{a3} Peak drift; ^{a4} Ultimate drift; ^{b1} Cracking load; ^{b2} Yield load; ^{b3} Peak load; ^{b4} Ultimate load; ^{c1} Cracking inter-story drift rotation of Δ_{cr}/h_f ; ^{c2} Yield inter-story drift rotation of Δ_y/h_f ; ^{c3} Peak inter-story drift rotation of Δ_p/h_f ; ^{c4} Ultimate inter-story drift rotation of Δ_u/h_f . h_f is the floor height.
^e Abbreviation of the ductility index; ^f Abbreviation of the average ductility index of both the positive and negative ductility index.

4.4.4. Ductility Index and Inter-Story Drift Rotation

The ductility is typically expressed using a ductility index. The ductility index β is the ratio of the ultimate drift Δ_u to the yield drift Δ_y of the envelope curve [8].

$$\beta = \Delta_u / \Delta_y \quad (1)$$

Table 5 indicates that the ductility index for panel-assembled walls ranges from 8.55 to 14.01, significantly exceeding the ductility index of 2.8 for concrete frames. This suggested that panel-assembled walls remained in a safe state before the main frame structure failed. The door frame could lead to a reduction in the ductility of the panel-assembled walls. The average ductility index of specimen W2 was 20.64% higher than that of specimen W1.

According to the Chinese standard of Code for Seismic Design of Buildings (GB 50011-2010) [12], the limit value of elastic inter-story drift rotation for the R.C. frame is 1/550, and the limit value of elastic–plastic inter-story drift rotation of the R.C. frame is 1/50. Based on Table 5, the values of the elastic inter-story drift rotation of the panel-assembled walls ranged from 1/200 to 1/147, which was greater than 1/550. The values of elastic–plastic inter-story drift rotation, 1/17 to 1/14, were greater than 1/50, meeting the code requirements. Furthermore, it was important to note that the values of the cracking inter-story drift rotation of panel-assembled walls was 1/330, exceeding 1/550, and the values of the peak inter-story drift rotation were 1/41 to 1/27, exceeding 1/50. These findings suggest that the panel-assembled walls retained their elasticity during frequent earthquake activity. Additionally, under rare earthquake conditions, they had not yet reached their peak state, indicating a substantial safety margin.

4.4.5. Strength Degradation

Strength degradation corresponds to the strength decrease induced by the subsequent cycle for a given displacement [8]. The strength degradation factor is given as:

$$\lambda_{ij} = F_j^2 / F_j^1 \quad (2)$$

where λ_{ij} is the strength degradation factor for the cycle 'i' with the intended displacement 'j', F_j^1 is the load (in kN) undertaken by the structure at the first cycle with the intended displacement 'j', and F_j^2 is the load (in kN) undertaken by the structure at the cycle '2' with the intended displacement 'j'.

As depicted in Figure 13, the strength degradation factor remained relatively high with an increase in the ductility ratio Δ/Δ_y , indicating a robust load-bearing capacity for the specimens. The positive strength degradation factor varied within the range of 0.71 to 0.98. For specimen W1, when the ductility ratio was less than 0.45 (corresponding to a displacement of 10 mm), the negative strength degradation factor was 0.68. Similarly, for specimen W2, when the ductility ratio was less than 1.80 (displacement less than 30 mm), the negative strength degradation factor ranged from 0.68 to 0.78. Furthermore, within the drift range of 40 mm to 150 mm, the negative strength degradation factor exceeded 0.82.

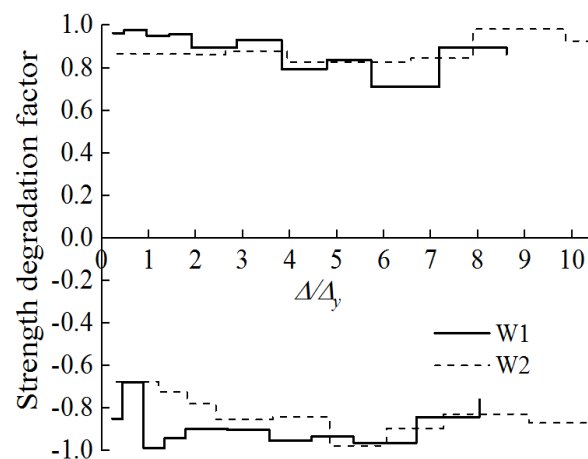


Figure 13. Strength degradation curves.

4.4.6. Stiffness Degradation

The stiffness of a structure is referred to as the secant stiffness (or effective stiffness) corresponding to a pair of displacement and force at a given cyclic amplitude [8]. The effective stiffness is justified as:

$$k_j^i = \sum_{i=1}^n F_j^i / \sum_j \Delta_j^i \quad (3)$$

in which k_j^i is the secant stiffness (in kN/mm) for the cycle 'i' with the intended displacement 'j'. F_j^i is the lateral loads (in kN) for the cycle 'i' with the intended displacement 'j'. And Δ_j^i is the drift (in mm) for the cycle 'i' with the intended drift 'j'.

As shown in Figure 14, there was minimal difference between the positive effective stiffness of specimen W1 and W2, as well as between their respective negative effective stiffness. This suggested that the door frame could enhance the lateral stiffness of the AAC panel walls, although its impact was relatively small.

As illustrated in Figure 14, the positive effective stiffness of the specimen exceeded its negative counterpart, and the effective stiffness decreased with an increase in the ductility ratio. When the ductility ratio was less than 2 (corresponding to a displacement of less than 30–40 mm), the positive effective stiffness decreased rapidly with an increasing ductility ratio. For specimen W1, the positive effective stiffness dropped from 3.3 kN/mm to

0.88 kN/mm, and for specimen W2, it decreased from 2.9 kN/mm to 0.95 kN/mm. When the ductility ratio was less than 1 (displacement less than 20 mm), the negative effective stiffness decreased rapidly with an increasing ductility ratio. For specimen W1, the negative effective stiffness decreased from 1.63 kN/mm to 0.58 kN/mm, and for specimen W2, it decreased from 1.43 kN/mm to 0.58 kN/mm. As the ductility ratio increased from 2 to 6, the effective stiffness gradually decreased. When the ductility ratio exceeded 6, the effective stiffness was close to a constant value of 0.14 kN/mm.

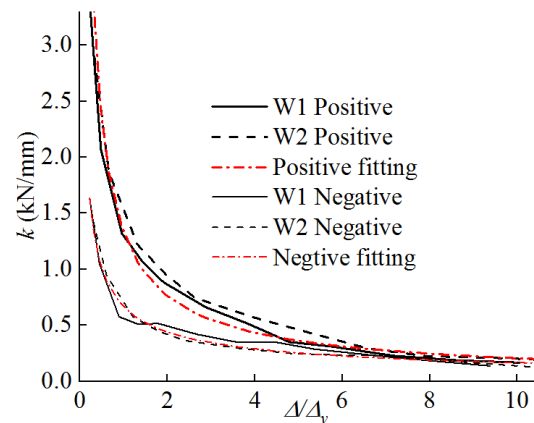


Figure 14. Stiffness degradation curves.

The relationship between the effective stiffness of specimens and the ductility ratio is represented by the fitting formula of Equation (4). The fitting curves are depicted in Figure 14, and it can be observed from the figure that the fitting curves align well with the experimental curves. The correlation coefficient (R^2) for the fitted Equation (4) is 0.98, indicating that the chosen curve model is well-suited:

$$\begin{aligned} k^+ &= 1.331 (\Delta/\Delta_y)^{-0.809} \\ k^- &= 0.665 (\Delta/\Delta_y)^{-0.600} \end{aligned} \quad (4)$$

where k^+ is the positive effective stiffness, and k^- is the negative effective stiffness.

4.4.7. Energy Dissipation

Energy dissipation is measured by the area enclosed by the load-drift hysteretic loop (shown in Figure 15). The area of hysteretic loops reflects the amount of energy dissipation. The larger the hysteretic loop, the better the energy dissipation capacity of a structure [8]. The energy dissipation capacity, which is denoted by the energy dissipation factor, is an important index to evaluate the structural seismic-resistance capacity. The energy dissipation factor is expressed as:

$$h_e = \frac{1}{2\pi} \cdot \frac{S_{abc} + S_{cda}}{S_{obe} + S_{odf}} \quad (5)$$

where h_e is the energy dissipation factor, $S_{abc} + S_{cda}$ is the area (in kN·mm) enclosed by the curves of ab-bc-cd-da. $S_{obe} + S_{odf}$ is the area (in kN·mm) enclosed by the polyline of o-b-e and o-d-f.

The energy dissipation factors for the specimens are depicted in Figure 16, and it can be observed from the figure that the energy dissipation factor varies within the range of 0.075 to 0.128, and the energy dissipation factor of specimen W1 was greater than that of specimen W2, indicating that the door frame could improve the energy dissipation capacity of the panel-assembled walls.

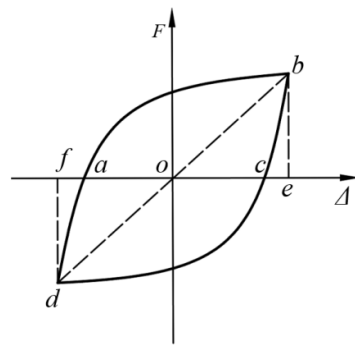


Figure 15. Hysteretic loop.

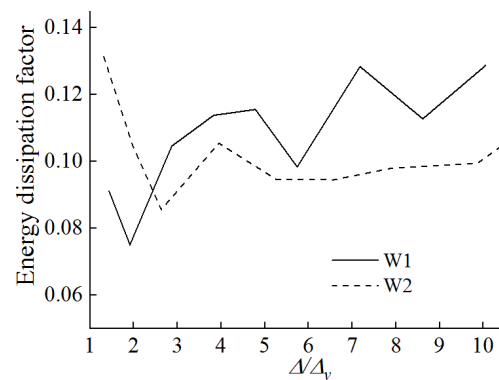


Figure 16. Energy dissipation factors.

5. Shear Angle

As depicted in Figure 6, the diagonal displacements of the assembled panels were measured using wire displacement gauges L-1 and L-2 [8]. As indicated in Figure 17, the shear deformation Δ_s in the assembled panels can be calculated using Equation (6). The shear angle is determined by Equation (7).

$$\Delta_s = \frac{1}{2} (\sqrt{(d_1 + D_1)^2 - h^2} - \sqrt{(d_2 + D_2)^2 - h^2}) \quad (6)$$

$$\gamma = \Delta_s / h \quad (7)$$

where, h represents the initial vertical distance between the ends of the wire displacement gauges, and L is the initial horizontal distance between these ends. Δ_s denotes the shear deformation, and γ is the shear angle. Additionally, d_1 and d_2 are the initial lengths of the diagonal lines, while D_1 and D_2 represent the measured values obtained from wire displacement gauges L-2 and L-1, respectively.

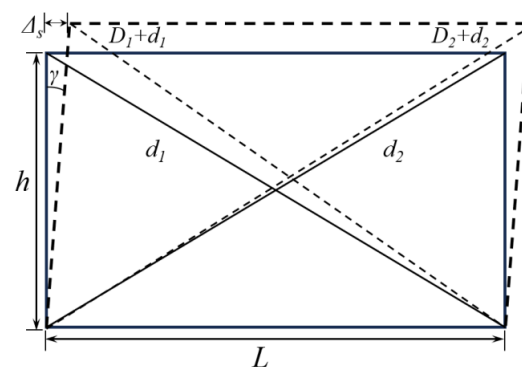


Figure 17. Shear deformation.

The correlation between the shear angle (γ) and the inter-story drift rotation (θ) is illustrated in Figure 18. It can be observed from the graph that the shear angle increased with the increase of inter-story drift rotation. The experimental data were fitted using a linear function, given by Equation (8), with a high correlation coefficient (R^2) of 0.99. The fitting results are depicted in Figure 18.

$$\gamma = 0.954\theta \quad (8)$$

where θ is the inter-story drift rotation.

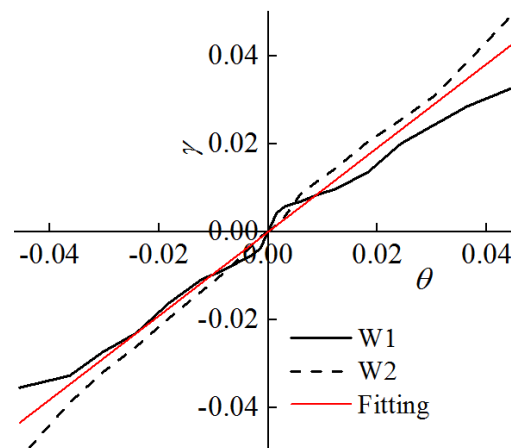


Figure 18. Shear angel.

6. Recovery Force Model

6.1. Envelope Model

Utilizing the methodologies from the hysteresis model discussed in references [21,22], this research employed a fitting approach and relied on experimental results to derive the normal envelope curve suitable for panel-assembled walls, as depicted in Figure 19. The expression for this curve is provided below:

$$\begin{aligned} y &= \alpha_1 x^{\alpha_2} & |x| \leq 1 \\ y &= \frac{x}{\eta(x-1)^2 + x} & |x| > 1 \end{aligned} \quad (9)$$

where $y = F/F_p$, $x = \Delta/\Delta_p$, Δ is the drift, Δ_p is the peak drift, F is the horizontal load, and F_p is the peak horizontal load. The parameters α_1 , α_2 , and η are given in Table 6.

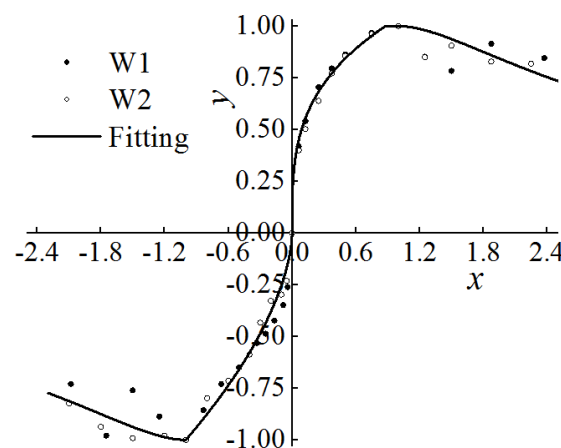


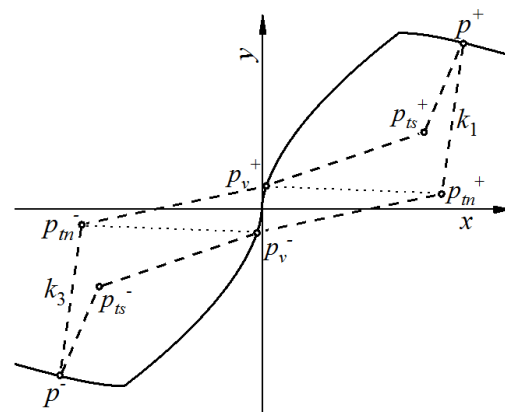
Figure 19. Theoretical envelope curve.

Table 6. Parameters for hysteresis curves.

Parameter	Value	
	Positive	Negative
α_1	1.040	0.930
α_2	0.302	0.448
β_{tn}	−0.0035	−0.011
c_{tn}	0.105	−0.11
β_D	0.283	0.118
c_D	0.019	0.077
μ_{tsx}	0.421	0.624
μ_{tsy}	0.349	0.367
k_{ini}	22.2 kN/mm	5.6 kN/mm
η		0.400
β_{pv}		0.111

6.2. Hysteresis Rule

The pivotal point model has exhibited extensive adaptability to various hysteresis models [23,24]. Drawing upon the features of the pivotal point model and the hysteresis loops of panel-assembled walls obtained, an eight-line-segment hysteresis rule is introduced, as depicted in Figure 20.

**Figure 20.** Hysteresis rule.

In Figure 20, the points p_v^+ and p_v^- represent pivot points situated on the skeleton curve. The relationship between the ordinate y_{pv} of the pivot point and the peak normal lateral load value y_p is illustrated by:

$$y_{pv} = \beta_{pv} y_p \quad (10)$$

where y_{pv} is the vertical coordinate of pivotal point, y_p is the peak point, and is equal to 1. And β_{pv} is a parameter, which is given in Table 6.

In Figure 20, the segments $p_v^+ - p_{tn}^+$ and $p_v^- - p_{tn}^-$ represent turning lines, and the points p_{tn}^+ and p_{tn}^- on these lines serve as turning points for the unloading curves. The unloading curve undergoes a directional shift upon intersecting the turning line. The expression for the turning line is detailed in Equation (11):

$$y_{tn} = \beta_{tn} x_{tn} + c_{tn} \quad (11)$$

where β_{tn} and c_{tn} are the parameters given in Table 6.

The curves $p_{tn}^- - p_v^+ - p_{ts}^+ - p^+$ and $p_{tn}^+ - p_v^- - p_{ts}^- - p^-$ are the loading curves, while $p^+ - p_{tn}^+$ and $p^- - p_{tn}^-$ represent the unloading curves. The unloading stiffness for segment $p^+ - p_{tn}^+$ is denoted as k_1 , and for segment $p^- - p_{tn}^-$, it is represented as k_3 :

$$k_i = (1 - D)k_{ini} \quad (12)$$

where k_i represents both k_1 and k_3 , with k_1 denoting the stiffness during positive unloading and k_3 representing the stiffness during negative unloading. k_{ini} signifies the stiffness at the pivotal points p_v^+ and p_v^- . D stands for the damage factor, and its expression is defined as follows:

$$D = \beta_D x + c_D \quad (13)$$

where β_D and c_D are parameters, which are given in Table 6.

The points p_{ts}^+ and p_{ts}^- denote the turning points of the loading curves, with coordinates (x_{ts}, y_{ts}) . The coordinate expressions for the turning points of the loading curves are as follows:

$$\begin{aligned} x_{ts} &= \mu_{tsx} x_i \\ y_{ts} &= \mu_{tsy} y_i \end{aligned} \quad (14)$$

where x_i is normal intended drift, y_i is normal intended horizontal load, and μ_{tsx} and μ_{tsy} are parameters given in Table 6.

To validate the accuracy of the proposed recovery force model, a hysteresis model generation program was developed. Through comparison, it was found that the generated hysteresis curve aligns well with the loading and unloading curves, turning points, and other key features of the specimen's hysteresis curves.

7. Finite Element Analysis

The finite element (FE) analysis of specimens was conducted using the commercial software ABAQUS of the version number 6.10 (Dassault Systems Simulia Corp., Providence, RI, USA). Solid elements of C3D8R were employed for the confining frame and AAC panels [25]. Concrete and AAC instances were modeled using a concrete damage constitutive model.

The stress–strain curve of concrete under uniaxial compression is as follows:

$$\sigma = (1 - d_c) E_c \varepsilon \quad (15)$$

where d_c is the compressive damage factor and can be determined according to literature [14].

The stress–strain curve of concrete under uniaxial tension is as follows:

$$\sigma = (1 - d_t) E_t \varepsilon \quad (16)$$

where d_t is the tensile damage factor and can be determined according to literature [14].

Plastic damage parameters of concrete are shown in Table 7. In Table 7, f_{b0}/f_{c0} is the ratio of biaxial ultimate compressive strength to uniaxial ultimate compressive strength.

Table 7. Plastic damage parameters of concrete.

Dilation Angle	Eccentricity	f_{b0}/f_{c0}	Viscosity Parameter
30°	0.1	1.667	0.005

Steel bars in prefabricated components and panels were simulated using truss elements, and the constitutive model for steel reinforcement employed a bilinear hardening model. In the elastic range, the modulus of elasticity for the steel reinforcement is $E_s = 210,000$ MPa, and the hardening modulus for the steel reinforcement is $E_s = \alpha E_s$, where $\alpha = 0.01$. The stress–strain relationship for the steel reinforcement is as follows:

$$\sigma = \begin{cases} E_s \varepsilon & 0 \leq \varepsilon \leq \varepsilon_y \\ f_y + (\varepsilon - \varepsilon_y) \alpha E_s & \varepsilon_y \leq \varepsilon \leq \varepsilon_u \end{cases} \quad (17)$$

The connection between steel reinforcements and concrete was modeled using an embedded contact relationship.

The connection between AAC panels was simulated using a face-to-face contact relationship with a friction coefficient of 0.35. Similarly, the connection between AAC panels and prefabricated components also utilized a face-to-face contact relationship with a friction coefficient of 0.45.

The L-shaped connectors between the AAC panels and the confining frame were modeled using three-directional spring elements. The bottom boundary conditions of the specimen were set as fixed boundaries. Cyclic loading was applied to the coupling nodes at the top of the specimen. The installation gap between the AAC panels and prefabricated components was set at 20 mm. The fixed boundary was applied to the bottom of the base of the models.

The concrete components and AAC panels were modeled using solid elements, with the element type of C3D8R. The rebar was modeled using three-dimensional truss elements of T3D2. The element size for the concrete components was 50 mm, while for the AAC panels it was 100 mm. The rebar element size was 50 mm. This mesh division balanced the computation speed and accuracy in the Abaqus numerical simulation software.

Through FE analysis, it was observed that between the yield load and the peak load, the specimen had undergone damage in the confining frame, with no failure observed in the AAC panels. With an increase in drift, the interaction between the panels and the confining frame strengthened, causing the top beam of the confining frame to bend upward, generating tensile stress at the upper part of the top beam.

At a drift of 210 mm, the maximum principal stress in the frame of specimens W1 and W2 is shown in Figures 21a and 21b, respectively. The Mises stress in the AAC panels of specimens W1 and W2 is shown in Figures 21c and 21d, respectively.

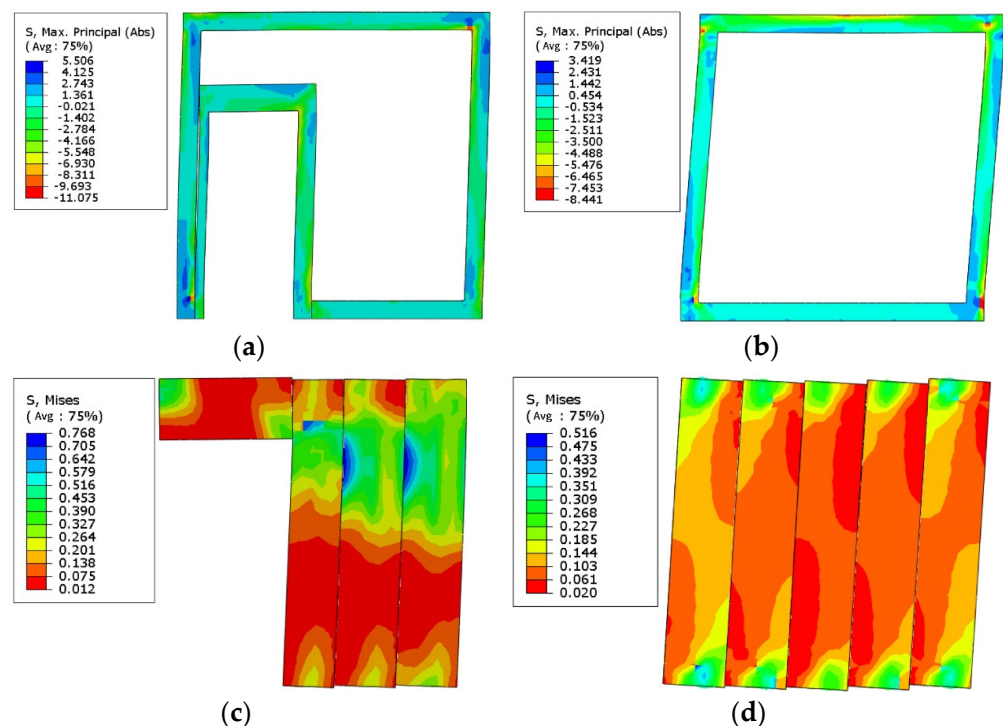


Figure 21. Stress distribution. (a) Confining frame of W1, (b) Confining frame of W2, (c) Panels of W1, (d) Panels of W2.

A comparison between Figures 8 and 21 indicated that the deformation patterns derived from FE analysis for the confining frames and AAC panels aligned with those observed in experiments, confirming an interaction between the wall panels and the frames. Stress concentrated at the corners of the confining frames and the AAC panels. When the diagonal strut effect of the AAC panels reached its critical limit, shear failure occurred at the corners of the panels. Furthermore, the specimen without a door frame primarily transmitted pressure from the confining frame to the AAC panels, while the specimen with a door frame could transmit pressure through the door frame to the AAC panels.

The hysteresis curves obtained from the simulation and tests for specimens W1 and W2 are shown in Figures 22a and 22b, respectively. The envelope curves from both the simulation and tests for specimens W1 and W2 are presented in Figures 22c and 22d, respectively.

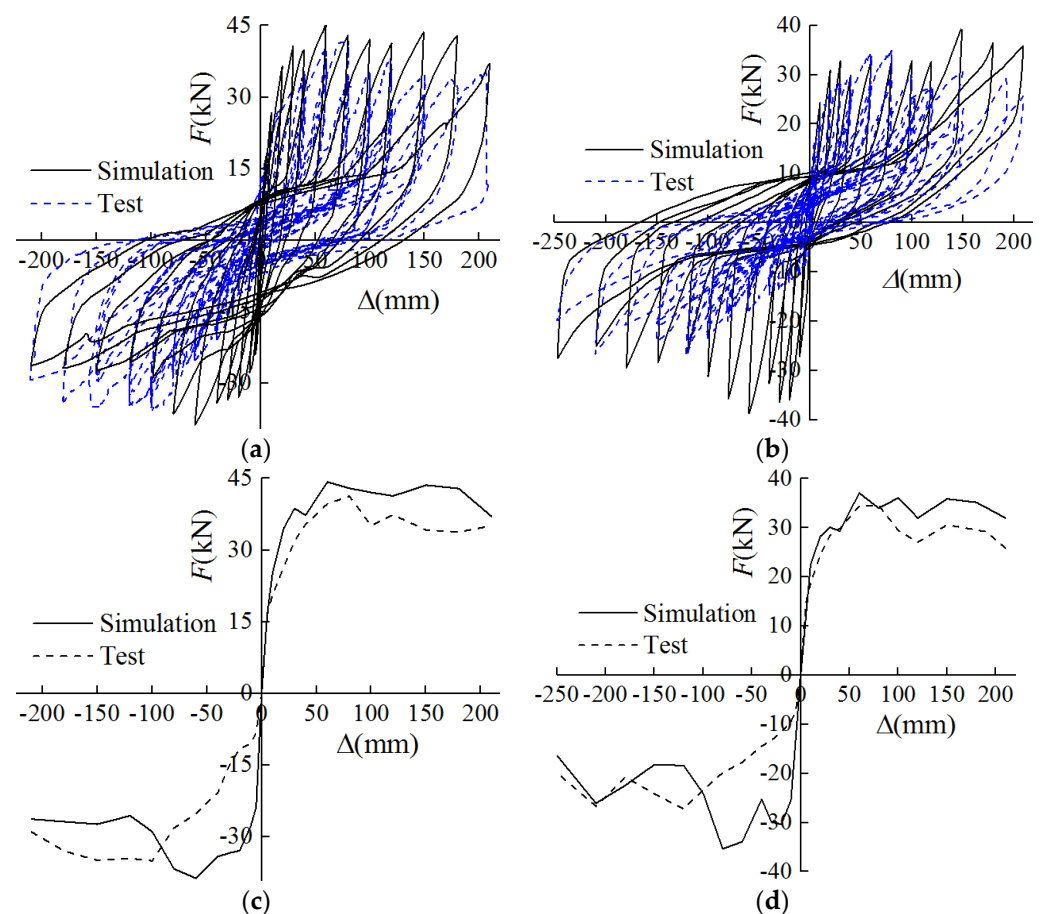


Figure 22. Comparisons of simulation and test results. (a) Hysteresis curves of W1, (b) hysteresis curves of W2, (c) envelope curves of W1, (d) envelope curves of W2.

A comparison between the simulated and experimental hysteresis and envelope curves revealed the following: FE analysis could capture the pinching effect in the hysteresis curves of the AAC panel-assembled walls. In the positive direction, the FE generated hysteresis and envelope curves closely matched the experimental results. In the negative direction, when the drift exceeded the peak drift, the agreement between the simulated and experimental curves was also strong. However, when the drift in the negative direction was below the peak drift, deviations arose between the simulation and experimental outcomes.

The reason for this inconsistency was that when the drift was small, the lateral force was primarily borne by the confining frames, made of precast concrete, while the joints were cast-in-place; a discontinuity existed between the cast-in-place joints and the precast components. At a given drift, when the specimen experienced a drift in the positive

direction, it could cause damage at the discontinuity, reducing the specimen's capacity to resist lateral force in the negative direction. This discontinuity effect was difficult to simulate accurately with FE analysis.

8. Analytical Method of AAC Panel-Assembled Walls

8.1. Calculation Model

Based on the experimental results and finite element analyses, the failure sequence of AAC panel-assembled walls was as follows: firstly, the confining frames developed joints hinges; after that, the corners of AAC panels experienced shear failure sequentially from both sides toward the center.

For calculating the internal forces of confined AAC panel-assembled walls, the following assumptions were taken:

- The lateral load-bearing capacity of the confining frame follows an ideal elastic–plastic constitutive model;
- The failure load of AAC panels adopts a brittle fracture model;
- The diagonal strut effect of confined AAC panels is modeled using a single diagonal brace model, where each brace can only withstand compression, as shown in Figure 23a;
- The connections between the confining frame and AAC panels are hinged and the braces fail one after another, as shown in Figure 23a.

The calculation for shear failure at the corner of the diagonal brace is depicted in Figure 23b. In the process of the experiment, the lengths of the failure corners of the AAC panels were measured. After counting the geometrical dimensions of the 11 destroyed wedges, it was found that the average length of the side BC of the triangle ABC was 270 mm, and the average angle α in Figure 23b was 21.75° .

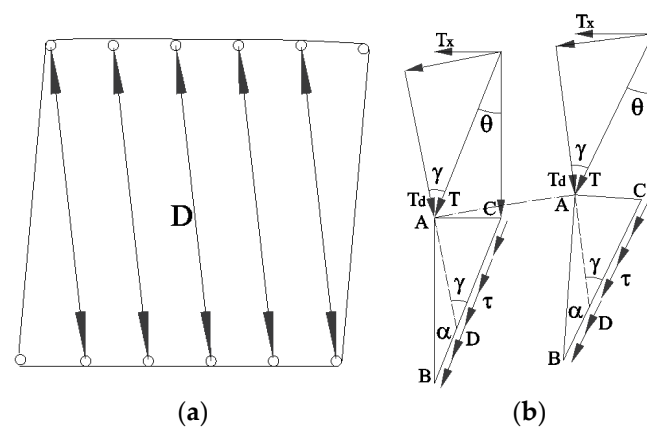


Figure 23. Diagonal strut effect. (a) Diagonal strut model, (b) corner failure analysis.

From Figure 23b, it is evident that when AAC panels undergo rotation along the lower right corner, the angle γ between the diagonal line AD and the inclined side BC of the triangle ABC remains constant at 33.06° . In the event of shear failure at the panel corner, the shear force T on the inclined side BC of right-angled triangle ABC is determined by:

$$T = \tau \times t \times l_{BC} \quad (18)$$

where τ is the shear strength of AAC and is 0.5 N/mm^2 (shown in Table 3), t is the thickness of AAC panels and is 200 mm (shown in Table 1), and l_{BC} is the average length of the side BC and is 270 mm.

During the rotational motion of the panel, the component force T_d along the diagonal line AD of the resultant force T remains constant. During the occurrence of shear failure at the corner of the panel, the critical limit value T_d for the AAC panel failure can be derived according to the following formula:

$$T_d = T \times \cos\gamma \quad (19)$$

In the single brace model, the horizontal force corresponding to the failure of diagonal brace D is calculated using the formula:

$$F_d = T_x = T \times \sin(\theta) \quad (20)$$

As the angle θ increases from 0 °C to 5 °C, the drift increases from 0 mm to 263.75 mm, and the horizontal force F_d increases from 10 kN to 12.15 kN; the average force F_d is 11.08 kN. In engineering applications, taking the F_d as 11.08 kN cannot introduce too much error. The shear failure parameters of an AAC panel are given in Table 8.

Table 8. Shear failure parameters.

τ	t	l_{BC}	α	γ	θ	T	T_d	F_d
N/mm ²	mm	mm	°	°	°	kN	kN	kN
0.5	200	270	21.75	33.06	11.31	27	22.63	11.08

8.2. D-Value Method

The calculation diagram of specimen W1 using the *D*-value method is illustrated in Figure 24a. In Figure 24a, the beam at the upper part of the door frame is connected to the confining frame through two fictitious members, assuming a hinged connection at the joints. The calculation diagram of specimen W2 using the *D*-value method is illustrated in Figure 24b.

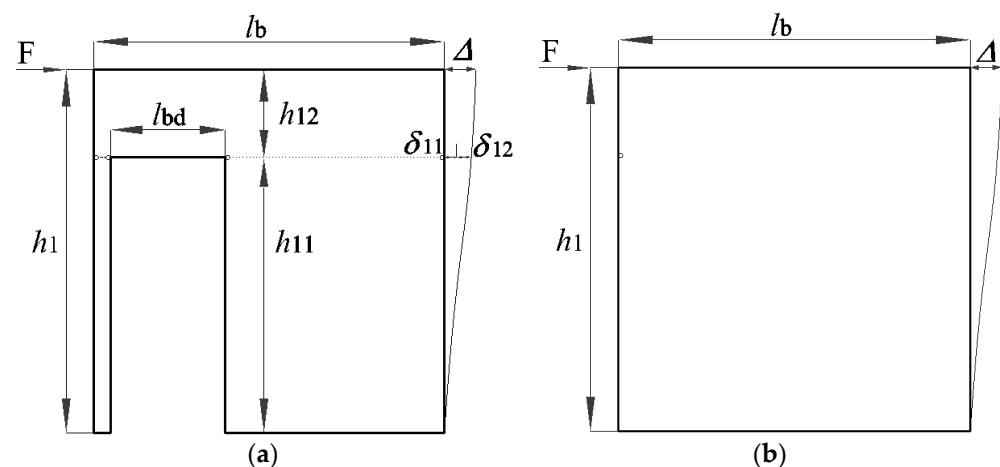


Figure 24. Calculation diagram. (a) W1 for *D*-value method, (b) W2 for *D*-value method.

The initial stiffness of the confining frame was calculated using the *D*-value method [26], and the results were compared with experimental findings. The experimental initial stiffness values in Table 9 represent the average of positive and negative initial stiffness (shown in Figure 14).

Table 9. Calculated initial stiffness.

	D_{ave}^a	D_{cal}^b	Error
	kN/m	kN/m	%
W1	3098	3722	16.75
W2	1525	1375	−10.93

^a Average experimental initial stiffness in the positive and negative direction; ^b Calculation of the initial stiffness using the *D*-value method.

As shown in Table 9, the calculated initial stiffness of the confining frame of the specimen W1 (W2) using the *D*-value method exhibiting a 16.75% (−10.93%) deviation with the average experimental initial stiffness in the positive and negative direction (shown in Figure 14). This value falls below the 20% threshold, making it acceptable from an engineering application perspective. This suggests that when displacements are small, the external forces acting on confined AAC panel-assembled walls are predominantly borne by the confining frame.

8.3. Yielding Resistance of Confining Frame

In accordance with the Chinese Code for Design of Concrete Structures (GB 50010-2010) [14], the calculation diagram for the yielding moment of the components in the confining frame is depicted in Figure 25a. When a column yields, the corresponding shear force at the flexure point of the column can be calculated based on moment equilibrium. The calculation diagram is illustrated in Figure 25b. The calculation diagram of shear force for a joint is presented in Figure 25c.

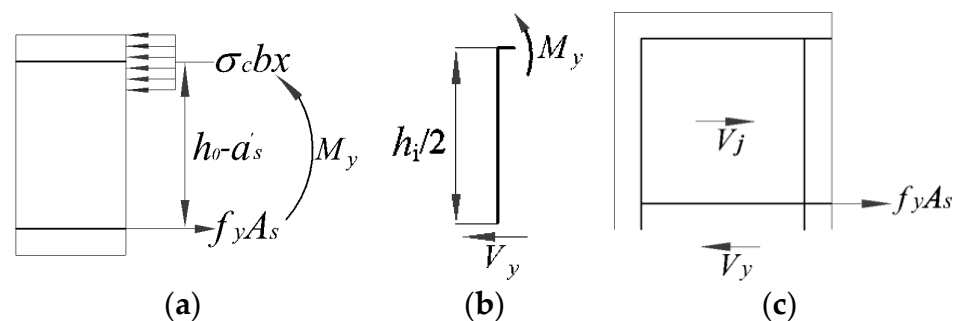


Figure 25. Calculation diagram. (a) Yielding moment, (b) shear force of column, (c) shear force of joint.

The formula for calculating the yielding moment is as follows:

$$M_y = f_y A_s (h_0 - a'_s) \quad (21)$$

where f_y is the yield tensile strength of reinforcements (shown in Table 3), A_s is the area of the reinforcements in the tensile region, and h_0 and a'_s are the effective height and the distance from the center of the bar to the tension face of the cross section of a concrete member, respectively.

The shear force corresponding to the yielding of the column is:

$$V_y = M_y / (h_i / 2) \quad (22)$$

where h_i is the length of the column.

The shear force sustained by a joint is calculated as:

$$V_j = f_y A_s - V_y \quad (23)$$

The horizontal force corresponding to the yielding of the confining frame is:

$$F_y = \sum_{i=1}^n V_{yi} \quad (24)$$

where V_{yi} is the yielding shear force of the column i and n is the number of columns.

The shear bearing capacity of a joint is:

$$V_{cs} = 0.7 f_t b h_0 + f_{yv} A_{sv} h_0 / s \quad (25)$$

where f_{yv} is the yield tensile strength of stirrups, f_t is the tensile strength for concrete (shown in Table 3), A_{sv} is the gross area of the multiple legs of stirrup, and s is the interval between stirrups. The parameters can refer to Table 3 and Figure 5.

The yielding force of columns F_y , the shear force V_j and the shear bearing capacity V_{cs} of the joints calculated according to Equations (23)–(25) are given in Table 10. At yielding, the shear force of the joints is relatively high, approaching the shear resistance capacity of the joints. When the AAC panels interact with the confining frame, the actual shear force at the joints will be even higher. This provides a force-based explanation for the formation mechanism of the joint hinges observed in the experimental process.

Table 10. Yielding resistance of confining frame.

	M_y kN × m	$h_i/2$ m	W1 F_y kN	V_j kN	V_{cs} kN	M_y kN × m	$h_i/2$ m	W2 F_y kN	V_j kN	V_{cs} kN
C _{dl} ^{a1}	3.5	1.225	2.86	11.84	36.80	--	--	--	--	--
C _{dr} ^{a2}	12.7	1.225	10.37	43.01	89.37	--	--	--	--	--
C _{ll} ^{a3}	12.7	1.650	7.70	84.36	89.37	12.7	1.650	7.70	84.36	89.37
C _{lr} ^{a4}	12.7	1.650	7.70	84.36	89.37	12.7	1.650	7.70	84.36	89.37
Sum	--	--	28.63	--	--	--	--	15.40	--	--

^{a1} Left column of the door frame; ^{a2} Right column of the door frame; ^{a3} Left column of the confining frame; ^{a4} Right column of the confining frame.

8.4. Lateral Load-Bearing Capacity of AAC Panel-Assembled Walls

The formula for calculating the peak load of the AAC panel-assembled walls is presented below:

$$F_w = F_y + F_d + F_p \quad (26)$$

where F_y is the yielding force of the confining frame, F_d is the ultimate support force of the diagonal brace, and F_p is the horizontal force needed to initiate panel rotation, overcoming the self-weight of the panels:

$$F_p = \sum_{i=1}^n G_{pi} W_{pi} / 2h_{pi} \quad (27)$$

where n represents the number of panels, W_{pi} is the width of the wall panel, G_{pi} is the self-weight of the panel i , calculated as $G_{pi} = V_{pi} \times \rho$ in kN, V_{pi} is the volume of the panel i , and ρ is the bulk density of the AAC panel and is equal to 0.8 kN/m³.

Table 11 presents the calculated peak load of the AAC panel-assembled walls based on Equation (26). F_{ave} in Table 11 represents the average values of the experimental positive and negative peak loads (refer to Table 5). The maximum error between the calculated and the experimental average peak load is 9.65%, less than 20%, which is deemed acceptable in engineering applications.

Table 11. Calculated bearing capacity.

	F_y kN	D_{cal} kN/m	Δ_y mm	F_{p3} ^c kN	F_{p2} ^b kN	$n \times F_{p1}$ ^a kN	F_d kN	F_w kN	F_{ave} kN	Error %
W1	28.63	3722	7.69	1.35	0.20	$2 \times 0.288 = 0.58$	11.08	41.83	38.40	8.94
W2	15.40	1375	11.2	--	--	$5 \times 0.288 = 1.44$	11.08	27.92	30.90	−9.65

^a Horizontal force needed to overcome the self-weight of the panel P₁; ^b horizontal force needed to overcome the self-weight of the panel P₂; ^c horizontal force needed to overcome the self-weight of the panel P₃ (as shown in Figure 3).

9. Conclusions

In this study, two full-scale specimens, one with a door opening and the other without, were fabricated and tested. The seismic performance of the panel-assembled walls was studied by experiments and FE analyses. Based on the test results and the FE analyses, the following conclusions could be developed:

1. The panels in the panel-assembled walls system played a crucial role in contributing to the lateral resistance. The diagonal strut effect of AAC panels further enhanced the system's ductility, energy dissipation capacity, and deformation capability.
2. The lateral load was initially borne by the frame when the drift was less than 20 mm, but as the drift increased, the interaction between the frame and panels intensified. After reaching the peak load, severe damage occurred at the corners of the frame, leading to the formation of 'joint hinges'. Subsequently, the primary responsibility for bearing the lateral load shifted to the panels.
3. The value of elastic inter-story drift rotation of the panel-assembled walls ranged from 1/200 to 1/147, which was greater than 1/550. The values of elastic-plastic inter-story drift rotation, 1/17 to 1/14, were greater than 1/50, not only meeting the requirements outlined in the Chinese building code but also demonstrating a substantial safety margin. Notably, the panel-assembled walls maintained a secure state even after the main frame structure reached failure.
4. The door frame was found to enhance the initial stiffness, peak load and energy dissipation capacity of the panel-assembled wall, but could lead to a small reduction in ductility. Compared to the specimen without the door frame, the peak lateral load of the specimen with the door frame increased by 19.7–30.1%.
5. Cracks were observed initially on the prefabricated columns, followed by the upper prefabricated beam and at the cast-in-place joints and corners of the AAC panels.
6. The effective stiffness of specimens with and without a door opening was close. The positive effective stiffness of panel-assembled walls exceeded its negative, decreasing with an increase in drift.
7. The developed restoring-force model and the method presented for predicting the lateral load-bearing capacity of confined AAC panel walls were acceptable in engineering applications.

Unlike infill walls between frames, this study examined the lateral load resistance and deformation capacity of AAC panel-assembled partition walls that were directly installed on the floor outside the frames. The findings from this research are not suitable for direct application to infill walls between frames. Additionally, if the tie columns in the partition walls do not meet the requirements of relevant seismic codes, directly applying the results and conclusions from this study could lead to significant errors.

Author Contributions: Conceptualization, X.L., Q.Z. and Z.Z.; Methodology, X.L., Q.Z. and Z.Z.; Software and validation, D.M.; Formal analysis, X.L. and D.M.; Investigation, Q.Z., Z.Z., H.B. and Y.Y.; Writing—original draft and writing—review and editing, X.L.; Resources, supervision and project administration, H.B. and Y.Y. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the Natural Science Foundation of Sichuan Province, grant numbers 2022NSFSC0095 & 2022NSFSC0319; Natural Science Foundation of Southwest University of Science and Technology, grant number 21zx7121; and Science and Technology Planning Project of Ningbo City, grant number 2022T003.

Data Availability Statement: The data presented in this study are available on request from the corresponding author.

Conflicts of Interest: Authors Qidi Zhang and Han Bao were employed by the company of China Railway Eryuan Engineering Group Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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