

Article

Study on the Seismic Performance of Prefabricated Walls Under the Synergistic Effect of Different Connection Methods and Low-Carbon Materials

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Abstract

Dry-connected prefabricated composite walls have been increasingly used in steel structures owing to the development of building industrialization and the demand for rapid and efficient construction. However, the respective influences of dry-connection configuration and wall panel material on the seismic response of prefabricated composite walls require further clarification. To address this issue, full-scale quasi-static tests, continuum damage mechanics (CDM) analysis, and finite element simulations were conducted. Six specimens were tested under low-cycle reversed loading. The influence of connection type was evaluated by comparing MRC walls with U-type and Z-type connections, whereas the influence of wall panel material was evaluated by comparing MRC and SFC walls under U-type connections. For the tested MRC walls, the deformation capacity of the connections strongly influenced the failure mode. The U-type rigid connections provided limited deformation buffering, resulting in stress concentration at the wall ends, initial cracking at displacements of 8–10 mm, rapid crack propagation, and semi-brittle failure. In contrast, the Z-type flexible connections released deformation demand through the slip and rotation mechanisms of the slotted holes. The maximum tested displacements of the Z-type specimens were approximately 2.58–2.78 times the ultimate displacements of the corresponding U-type specimens. The Z-type specimens maintained stable load-carrying behavior with limited panel damage throughout the tested displacement range. Furthermore, a restoring-force calculation method was established based on CDM theory, and the predicted peak loads were generally consistent with the experimental results. This study identifies the load-transfer, deformation, and energy-dissipation characteristics of the tested connection–panel configurations and provides comparative evidence for the further development of prefabricated composite wall systems.

Keywords: dry connection; prefabricated composite wall; wall panel material; seismic performance; quasi-static test; finite element analysis



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1. Introduction

The rapid development of global building industrialization has promoted the widespread use of prefabricated steel structures in modern construction. These structures

offer advantages such as high construction efficiency, standardized production, and a high degree of industrialization [1]. Prefabricated composite wall panels serve as key enclosure and lateral force-resisting components in steel structures. Their connection configurations and material properties directly determine the overall seismic safety of the structure. Therefore, the seismic performance of steel structures has become an important research focus in structural engineering. Numerous studies have been conducted on this topic in recent years. Zhang et al. [2] evaluated the failure modes and seismic performance indicators of composite confined walls through low-cycle reversed loading tests. Zhao et al. [3] analyzed the slip energy dissipation mechanism of bolted wall panels. They proposed a simplified hysteretic model that accounts for stiffness degradation. Zhong et al. [4] clarified the failure modes and load-transfer mechanisms of monolithic precast systems through experimental tests. Recent research on coupled and hybrid-coupled shear walls has further shown that variations in structural parameters can alter damage distribution, lateral resistance, secant stiffness, energy dissipation, and ultimate drift under cyclic loading [5]. Although these structural systems differ from the dry-connected prefabricated composite walls investigated in the present study, the results highlight the importance of jointly considering damage state, stiffness degradation, deformation capacity, and energy dissipation when interpreting the cyclic response of wall systems.

Dry connections can avoid the drawbacks of traditional wet connections, such as extensive wet work and long construction periods. They are highly suitable for the rapid construction requirements of prefabricated buildings and have gradually become a research focus in this field. Biondini et al. [6] stated that the connection configuration directly determines the overall seismic capacity of precast structures. Zhang et al. [7,8] developed novel bolted joints for ultra-high-performance concrete (UHPC) prefabricated wall panels and evaluated their mechanical properties. Han et al. [9] investigated various dry connections to explore their structural performance, providing data support for engineering applications. However, early dry connections were mostly rigid. The deformation coordination capacity was insufficient, which made the wall panels prone to brittle failure under seismic action. This behavior was considered a major cause of structural damage during several strong earthquakes between 2008 and 2017 [10–12]. To address this issue, various novel dry connectors have been developed. Goudarzi et al. [13] evaluated the shear performance of Z-shaped steel plate connectors in sandwich wall panels. Nie et al. [14] confirmed that U-shaped connectors could improve the seismic performance of steel column weak-axis connections. Liu et al. [15] achieved post-earthquake functional recoverability of precast walls by developing prefabricated shear walls with bolted plate connectors. Vaghei et al. [16] investigated the deformation capacity and energy dissipation performance of prefabricated walls connected with U-shaped channel steel under reversed loading.

Alongside the optimization of connection configurations, alternative wall panel materials have attracted increasing attention in the context of reducing the environmental burden associated with conventional cement-based construction. Portland cement is recognized as an important source of anthropogenic carbon emissions [17,18]. Accordingly, inorganic-bonded bamboo composites [19], magnesium oxysulfate cement-based products [20], and straw fiber panels [21] have been investigated as alternative wall materials. Hay et al. [18] investigated the hydration, carbonation, and corrosion resistance of reactive magnesia-based composites. Azevedo et al. [22] evaluated the carbonation durability of magnesium oxysulfate cement-based fiber boards, while Liu et al. [23] examined their thermal and fire resistance. Regarding straw fiber wall panels, Che et al. [24] investigated the seismic performance of light steel–straw board composite systems, and Zhang et al. [25] reported favorable deformation and energy-dissipation behavior for light steel–straw composite walls with built-in steel plates. However, the actual environmental performance of

these materials depends on the adopted system boundary, raw-material sources, manufacturing processes, transportation, service life, and end-of-life treatment. Therefore, their environmental benefits should be verified through a dedicated life-cycle or embodied-carbon assessment.

Despite progress in connection optimization and the development of alternative wall panel materials, existing studies mostly focus on either connection configurations [26–28] or wall panel materials [29–33]. Few studies have comparatively examined the combined influence of connection configuration and wall panel material. Research on dry connections is often limited to the mechanical properties of the connectors or the connection performance of traditional concrete wall panels. Conversely, studies on alternative wall panel materials mainly concentrate on their basic material properties. Few systematic studies have investigated the compatibility between connection types and wall panel materials. Particularly for magnesium oxysulfate cement and straw fiber panels, their seismic responses when used with different dry connections require further clarification. This limitation restricts the engineering application of such systems. Ge et al. [34] investigated the synergistic load-transfer behavior between precast walls and bolted joints. However, the effects of multiple parameters on these composite systems require further clarification.

In this study, low-cycle reversed loading tests were conducted on six full-scale prefabricated composite wall specimens in accordance with the relevant Chinese standards [35–42]. The experimental program provided two controlled comparisons. The M-U and M-Z specimens were used to evaluate the influence of connection type for MRC wall panels, whereas the M-U and Q-U specimens were used to evaluate the influence of wall panel material under U-type connections. Failure modes, hysteretic responses, stiffness degradation, deformation capacity, and energy dissipation were investigated. Complementary theoretical and finite element analyses were conducted to interpret the observed load-transfer and damage mechanisms and to examine selected parameter trends. Because SFC panels with Z-type connections and repeated full-scale specimens were not included, the conclusions are restricted to the tested configurations. The present study focuses on the structural and seismic performance of the tested wall systems. No life-cycle assessment or embodied-carbon analysis was conducted; therefore, no quantitative conclusion is drawn regarding the environmental superiority of the MRC or SFC panels over conventional wall systems.

2. Experimental Program

2.1. Node Configuration and Working Principle

The U-type connectors were fabricated from Q345 steel. During construction, they were spot-welded on both sides of the panel joints at the upper and lower steel beams. The wall panels were then directly inserted into the U-shaped slots to form rigid connections. Adjacent panels were vertically spliced using a structural adhesive, thereby forming an integral wall panel. This configuration established a continuous load-transfer system composed of the steel frame, U-type connectors, and rigidly spliced wall panels. Owing to the rigid restraint provided by the U-type connectors, frame deformation and lateral loads were directly transferred to the wall panels. Since no effective deformation-buffering mechanism was available, high compressive stresses were generated at small lateral displacements. This stress concentration tended to induce early cracking and even global damage of the wall panels.

The Z-type flexible connection consisted of a Z-shaped connector, a square anti-loosening nut, an M10 hexagonal bolt, and washers. The steel connector was fabricated from Q345 steel plates with a thickness of 6 mm and a width of 50 mm, and its surface was electro-galvanized for corrosion protection. A 30 mm × 15 mm slotted hole was reserved in the Z-shaped connector as the key deformation-control component. This slotted hole

allowed in-plane rotation and relative slip of the connector. The joint was connected to the steel frame through continuous angle steel, forming a flexible load-transfer path consisting of the steel frame, angle steel, Z-type connector, and wall panel. Through the rotation and slip of the Z-shaped connector, frame deformation was partially released and part of the input energy was dissipated. As a result, the forced constraint on the wall panels was reduced, and the seismic deformation capacity of the wall system was improved. The connection details are shown in Figure 1.



Figure 1. Connection joint configuration.

2.2. Specimen Design

Six full-scale composite wall specimens were designed for the quasi-static tests. The experimental matrix was arranged to provide two paired comparisons rather than a complete factorial combination of all variables. For the evaluation of connection type, specimens M-U1 and M-Z3, as well as specimens M-U2 and M-Z4, had the same MRC panel material and corresponding geometric dimensions but different connection types. For the evaluation of wall panel material, specimens M-U1 and Q-U1, as well as specimens M-U2 and Q-U2, had the same U-type connection and corresponding geometric dimensions but different panel materials. No SFC specimen with a Z-type connection was included. Each configuration was represented by one full-scale specimen; therefore, the results are used to identify comparative response trends rather than to evaluate statistical significance.

Two wall panel materials were used: magnesium oxysulfate cement-based composite panels (MRC) and solid straw fiber cement panels (SFC). The connection configurations consisted of U-type rigid connections and Z-type flexible connections. The geometric dimensions included small walls measuring 2966 mm × 3000 mm and large walls measuring 4100 mm × 4200 mm. The wall assembly configurations are illustrated in Figure 2, and the specimen parameters are summarized in Table 1.

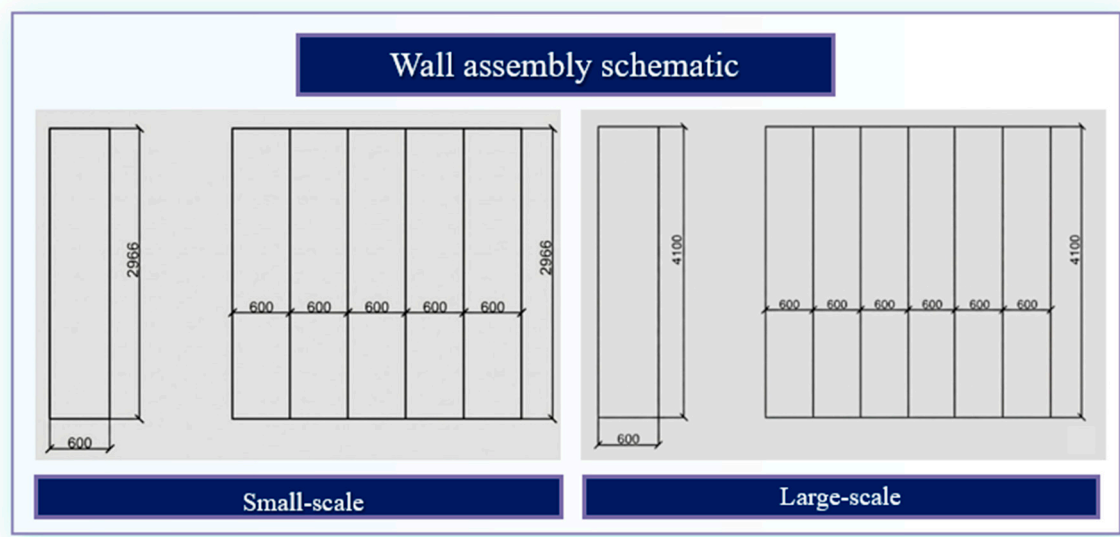


Figure 2. Schematic diagram of wall assembly (Unit: mm).

Table 1. Core parameters of wall specimens.

Specimen	Panel Material	Panel Specifications (mm)	Wall Dimensions (mm)	Assembly Method
M-U	MRC	2966 × 600 × 100	2966 × 3000 × 100	Vertical assembly of five strip panels
M-U2	MRC	2966 × 600 × 100	4100 × 4200 × 100	Vertical assembly of seven strip panels
M-Z3	MRC	2966 × 600 × 100	2966 × 3000 × 100	Vertical assembly of five strip panels
M-Z4	MRC	2966 × 600 × 100	4100 × 4200 × 100	Vertical assembly of seven strip panels
Q-U1	SFC	2966 × 600 × 100	2966 × 3000 × 100	Vertical assembly of five strip panels
Q-U2	SFC	2966 × 600 × 100	4100 × 4200 × 100	Vertical assembly of seven strip panels

2.3. Material Mechanical Properties

The steel used in the tests was Q345B-grade steel, with a nominal yield strength of 345 MPa and an elastic modulus of 2.06×10^5 MPa. According to design specifications [43], seven material property specimens were fabricated. To determine the material properties of the steel, data from two unrepresentative specimens were excluded, and the average values of the remaining five representative specimens were used. The universal testing machine is abbreviated as UTM hereafter. The detailed material properties are summarized in Table 2, where f_y is the yield strength, f_u is the ultimate strength, E_s is the elastic modulus of the steel, and δ is the elongation.

Table 2. Material parameters of the steel.

Statistical Parameter	f_y (N/mm ²)	f_u (N/mm ²)	E_s (N/mm ²)	δ (%)
Mean	370.4	480.4	201,250	25.39
SD	4.44	5.05	2674	0.66
COV	0.0120	0.0105	0.0133	0.0261

The mechanical properties of the MRC wall panel concrete were determined to provide reliable parameters for subsequent calculation and analysis. During the casting of the

components, three standard cubic specimens were reserved. These specimens were then tested using a compression testing machine. The tests were conducted in accordance with the Standard for Test Method of Mechanical Properties on Ordinary Concrete (GB/T 50081-2019) [44]. The testing apparatus is shown in Figure 3, and the test results are summarized in Table 3, where E_c is the elastic modulus of the concrete, f_c is the compressive strength, and $f_{cu,m}$ is the average compressive strength.



Figure 3. Steel tensile specimens and test instruments.

Table 3. Material parameters of the concrete.

Specimen Dimensions (mm × mm × mm)	Material Type	f_u (kN)	f_c (MPa)	$f_{cu,m}$ (MPa)	E_c (MPa)
150 × 150 × 150	MRC	469.1	20.8	19.96	2.2×10^4
		402.3	17.9		
		476.2	21.2		
150 × 150 × 150	SFC	9.6	6.4	6.3	814.5
		9.1	6.1		
		9.5	6.4		

2.4. Test Loading Device and Loading Protocol

The tests were conducted at the Sichuan Provincial Key Laboratory of Shock and Vibration of Engineering Materials and Structures at Southwest University of Science and Technology. The test setup is illustrated in Figure 4. Horizontal reversed loads were applied using an MTS electro-hydraulic servo loading system. This system has a maximum load capacity of ± 1000 kN and a maximum stroke of ± 150 mm. Vertical loads were uniformly applied to the top of the wall using a hydraulic jack and a steel spreader beam.

The tests were conducted using a displacement-controlled low-cycle reversed loading scheme. This scheme complied with the Specification for Seismic Test of Buildings (JGJ/T 101-2015) [42]. Before the wall cracked, the displacement increment for each loading level was 10 mm, and one cycle was applied at each level. After cracking, the displacement increment remained 10 mm, and three cycles were applied at each level. The test was

terminated when any of the following criteria were met: (1) The lateral load decreased below 85% of the peak load; (2) Through cracks developed or corner crushing occurred in the wall panels; (3) Local buckling or weld fracture occurred in the steel frame. The loading protocol is illustrated in Figure 5.

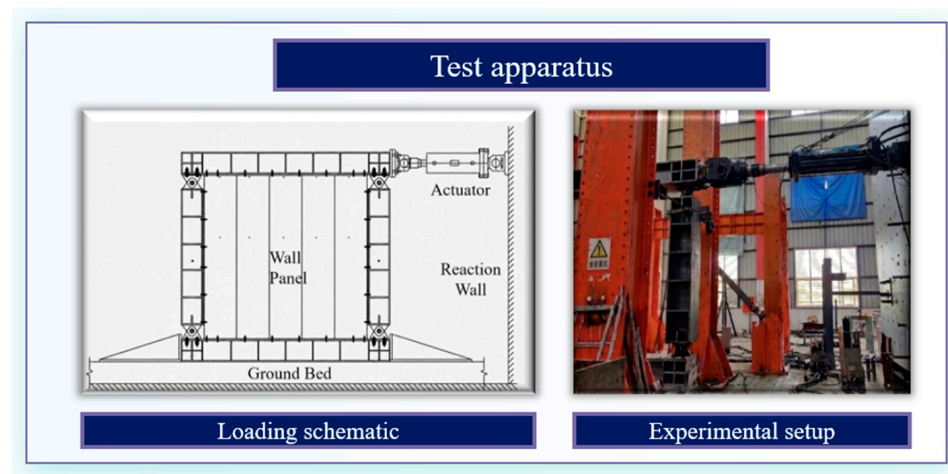


Figure 4. Test loading setup.

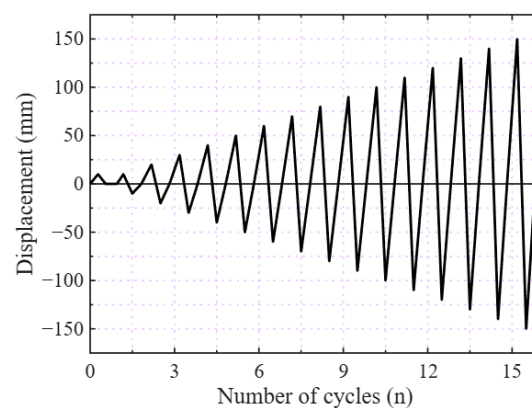


Figure 5. Test loading regime.

3. Test Phenomena and Results

3.1. Test Phenomena of Each Specimen

3.1.1. Damage Phenomenon

The local failure modes of each specimen are shown in Figure 6. For specimen M-U1, a crushing sound was heard from the mortar at the upper panel joints during preloading to 3 mm. When the displacement reached 10 mm in the first cycle of formal loading, a vertical crack appeared between the first and second wall panels from the west. This was accompanied by intermittent crushing sounds and minor mortar spalling. At a displacement of 15 mm, a diagonal crack appeared on the first western panel and propagated continuously. When the displacement reached 25 mm, numerous diagonal cracks developed on the wall panels. All vertical joints cracked; severe mortar spalling occurred, and the load-carrying capacity decreased rapidly. Finally, at a displacement of 60 mm, through-vertical cracks appeared. The load-carrying capacity dropped below 85% of the peak value, and the test was terminated.



Figure 6. Observed failure patterns, showing extensive cracking and corner crushing in the U-type walls and localized corner cracking in the Z-type walls.

It was observed that the first crack in specimen M-U2 appeared at the interface between the rightmost wall panel and the steel frame at a displacement of 8 mm. When the displacement reached 32 mm, the upper part of the vertical splicing joint at the top right corner cracked with a loud noise, and the corner of the panel was crushed. At a displacement of 48 mm, a 1.6-m-long vertical crack appeared at the interface between the leftmost panel and the frame. Numerous diagonal cracks developed at a displacement of 56 mm, and all splicing joints became fully cracked. The load-carrying capacity dropped below 85% of the peak value at a displacement of 60 mm, and the test was terminated.

For specimen M-Z3, no visible cracks appeared on the wall panels before the displacement reached 130 mm. The Z-type connections rotated normally with the wall panels without detaching. At a displacement of 130 mm (inter-story drift ratio of 1/23), minor cracks appeared at the corners of panel No. 5. Cracks subsequently appeared at the corners of panels No. 1 and No. 4 at a displacement of 135 mm. When the displacement reached 150 mm (inter-story drift ratio of 1/20), the cracks propagated slightly without penetrating. The Z-type connections remained intact throughout the process.

For specimen M-Z4, no visible cracks appeared before the displacement reached approximately 155 mm. At a displacement of 155 mm, corresponding to an inter-story drift ratio of approximately 1/26.4, localized cracks appeared at the corners of panels No. 1 and No. 5. A 450-mm-long crack was observed at the bottom-right corner of panel No. 4, while the remaining panels remained generally intact. The Z-type connectors showed no fracture or detachment and remained well connected throughout the test. For specimens M-Z3 and M-Z4, the tests were terminated because the actuator reached its displacement range limit.

At test termination, the prescribed structural failure criteria had not been reached: the wall panels exhibited only localized corner cracking, the Z-type connectors remained intact, and the lateral resistance had not decreased below the specified failure threshold. Therefore, the recorded terminal displacements are defined as the maximum tested displacements rather than the true ultimate displacements.

Specimen Q-U1 remained in the elastic stage during the initial loading phase. The hysteresis loops were spindle-shaped, and no visible cracks appeared. At a displacement of 10 mm, through-vertical cracks developed at the interfaces between both sides of the wall and the steel frame. When the displacement reached 20 mm, vertical cracks appeared at the splicing joints of the middle panels, and the load-carrying capacity reached its peak. At a displacement of approximately 25 mm, the vertical joints became fully cracked, diagonal cracks developed at the bottom of the panels, and local crushing occurred along the wall edges. When the displacement reached 25.63 mm, the load-carrying capacity decreased below 85% of the peak value, and the test was terminated. At this stage, fragments fell from the panel splicing joints.

For specimen Q-U2, a through-vertical crack appeared at the connection between the right side of the wall and the steel frame at a displacement of 16 mm. At a displacement of 25 mm, a vertical crack appeared at the splicing joints of the middle panels. When the displacement reached 48 mm, the vertical joints became fully cracked. Diagonal cracks appeared at the bottom of the panels due to compression. During the later loading stages, the cracks continued to develop. The edges of the wall were crushed and cracked, generating friction sounds against the steel frame. The test was ultimately terminated because the load-carrying capacity decreased to the prescribed limit.

3.1.2. Comparative Analysis of Failure Modes and Failure Mechanisms

A semi-brittle global failure mode was observed for the U-type rigid connection joints. The initial cracking displacement was 8 to 10 mm. The primary failure characteristics included through-cracks at the panel joints, diagonal cracks distributed across the wall, and corner crushing. No clear damage was observed on the connectors themselves during the failure process. The wall panels became the weak load-bearing zones of the entire system. This behavior is attributed to the lack of deformation buffering in the rigid load-transfer path. Lateral displacements of the frame were directly converted into concentrated stresses at the edges of the wall panels. Consequently, early cracking was triggered in the walls. Under low-cycle reversed loading, the cracks propagated rapidly, and the vertical joints became fully cracked. This process was accompanied by mortar spalling and local crushing, resulting in a sudden decrease in the load-carrying capacity. Furthermore, the out-of-plane constraints of the joints were weak. Therefore, the wall panels were prone to slipping and warping. Detachment from the U-shaped slots induced out-of-plane instability, which accelerated the global failure. Ultimately, the system exhibited a high initial stiffness, but the stiffness degraded rapidly. The load-carrying capacity decreased suddenly after the ultimate load was reached, indicating clear global brittle characteristics.

In contrast, the Z-type flexible connections exhibited large deformation capacity with limited damage within the tested displacement range. No structural failure state was reached because the tests were terminated by the actuator displacement limit. Visible panel cracking was delayed until displacements of 130–155 mm, and the observed damage was limited mainly to localized corner cracks. The connectors remained intact throughout the tests. The slip and rotation mechanisms of the slotted holes partially released the frame deformation demand and reduced, rather than completely eliminated, stress concentration in the wall panels. During reversed loading, panel damage developed gradually and remained localized, while global cracking, connector detachment, and out-of-plane wall

instability were not observed. The connections maintained effective in-plane load transfer and out-of-plane restraint, allowing coordinated deformation between the frame and wall panels. The initial stiffness was relatively low, and the subsequent stiffness reduction was gradual. Stable load-carrying behavior was maintained up to the maximum tested displacement. These observations demonstrate favorable large-deformation capacity within the investigated range; however, the true ultimate deformation capacity was not determined.

3.2. Test Results

3.2.1. Hysteretic Curve

The load–displacement hysteretic curves of the wall specimens are presented in Figure 7. The hysteretic curves of the M-U series exhibited relatively full spindle-shaped loops and generally good symmetry between the positive and negative loading directions, indicating stable cyclic response before severe panel damage occurred. Owing to the comparatively high strength of the MRC panels, the pinching effect was less pronounced than that of the SFC specimens. However, the rigid deformation constraint imposed by the U-type connections led to local panel crushing and a rapid decrease in lateral resistance during the later loading stages. The ultimate displacements of the M-U specimens ranged from 53.37 to 60.27 mm, indicating an overall semi-brittle response. In contrast, the Q-U specimens exhibited pronounced pinching and greater asymmetry between the positive and negative loading directions. For specimen Q-U2, the negative lateral resistance was approximately 20% lower than the positive resistance. Under the combined effects of rigid constraint, stress concentration, local crushing, and interfacial sliding, the ultimate displacements of the Q-U specimens were limited to 25.63–49.08 mm. Under the same U-type connection condition, the MRC specimens exhibited fuller hysteresis loops, higher peak resistance, and ultimate displacements approximately 23–108% greater than those of the corresponding SFC specimens.

The M-Z specimens exhibited slip-type hysteretic responses with long and relatively flat plateaus near the coordinate origin. Energy dissipation was mainly associated with connector slip, friction, and localized panel damage. As the displacement amplitude increased, the slip-dominated portion of the hysteresis loops gradually extended, while an increase in lateral resistance appeared at large displacements as the boundary clearances were progressively closed. The maximum tested displacements reached 148.34 mm for M-Z3 and 155.37 mm for M-Z4. Stable lateral resistance was maintained up to these displacements, and no abrupt strength loss or global wall failure was observed. These results demonstrate favorable large-deformation capacity within the tested range but do not represent the true ultimate ductility of the specimens.

For the tested MRC wall specimens, connection type strongly influenced the hysteretic response and damage mode. Although the Z-type specimens exhibited more pronounced pinching during the initial loading stages, connector slip and rotation enabled coordinated deformation between the frame and wall panels and reduced the panel deformation demand. Consequently, the Z-type specimens exhibited a wider stable deformation range and less severe wall damage within the tested range. Their maximum tested displacements were approximately 2.58–2.78 times the ultimate displacements of the corresponding U-type specimens. No abrupt decrease in lateral resistance was observed before the Z-type tests were terminated by the actuator displacement limit.

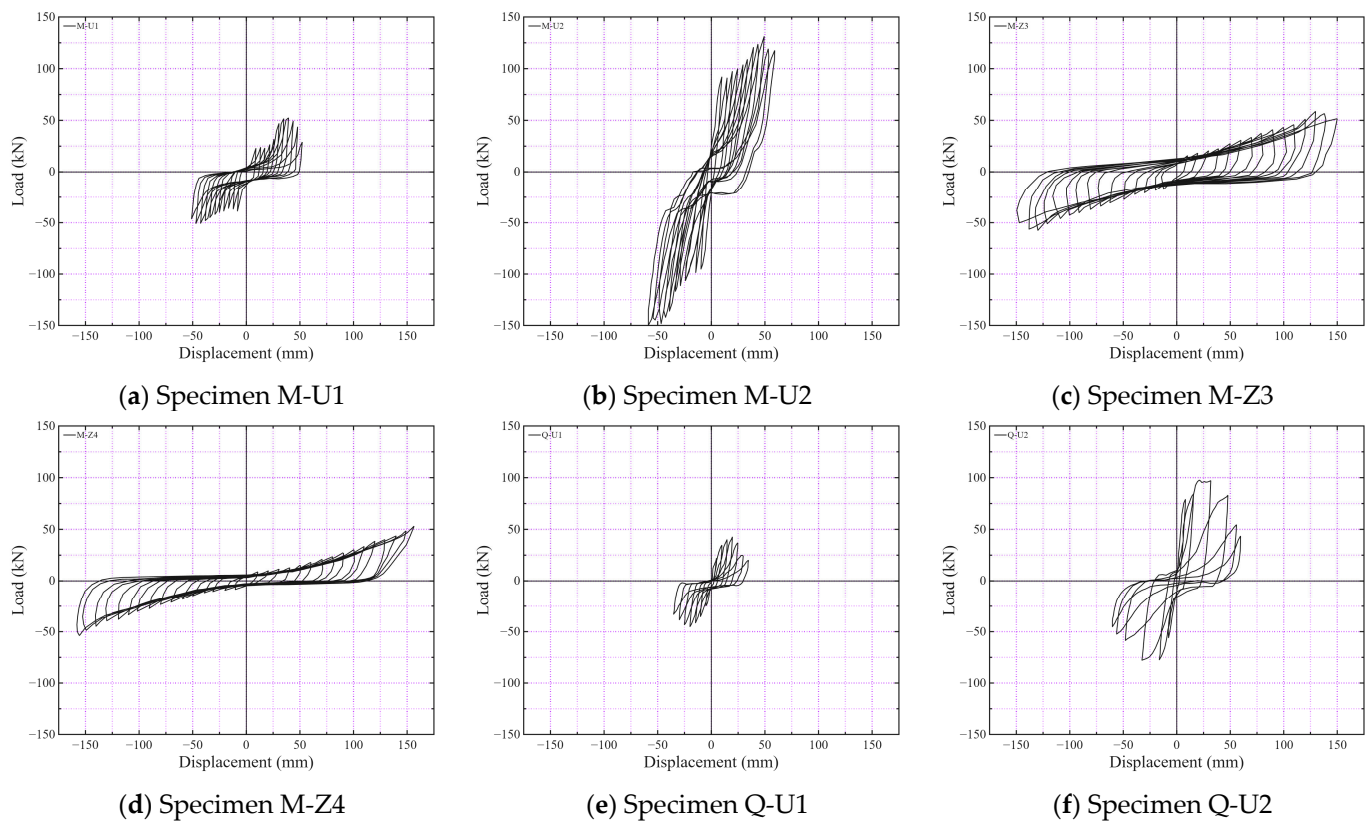


Figure 7. Hysteretic curves, showing fuller loops for the M-U series, slip-dominated pinching for the M-Z series, and marked degradation for the SFC series.

3.2.2. Deformation Capacity and Skeleton Curves

The load–displacement skeleton curves of the wall specimens are shown in Figure 8. The loads and displacements at each performance point are listed in Table 4. Δ_y is the yield displacement. For specimens that reached the prescribed failure criterion, Δ_u and P_u denote the ultimate displacement and corresponding failure load, respectively, and the displacement ductility coefficient is calculated as $\mu = \Delta_u / \Delta_y$. For specimens M-Z3 and M-Z4, structural failure was not reached. Therefore, Δ_e and P_e denote the maximum tested displacement and corresponding terminal load, respectively, and a conventional ultimate displacement ductility coefficient is not reported. The differences in load-carrying capacity and energy dissipation are analyzed based on the working mechanisms of the materials and connections. The coefficient μ is calculated by Equation (1):

$$\mu = \frac{\Delta_u}{\Delta_y} \quad (1)$$

For the M-U series, an approximately symmetric peak distribution was observed due to the high strength of the MRC material. The ultimate load was reached at a relatively small displacement. The ascending branch exhibited a rapid linear increase. After the peak load, a rapid decrease in the load-carrying capacity occurred due to the fast accumulation of damage. The ultimate displacement was limited to the range of 45 to 60 mm. The displacement ductility coefficient μ ranged from 1.96 to 3.47, indicating a semi-brittle failure.

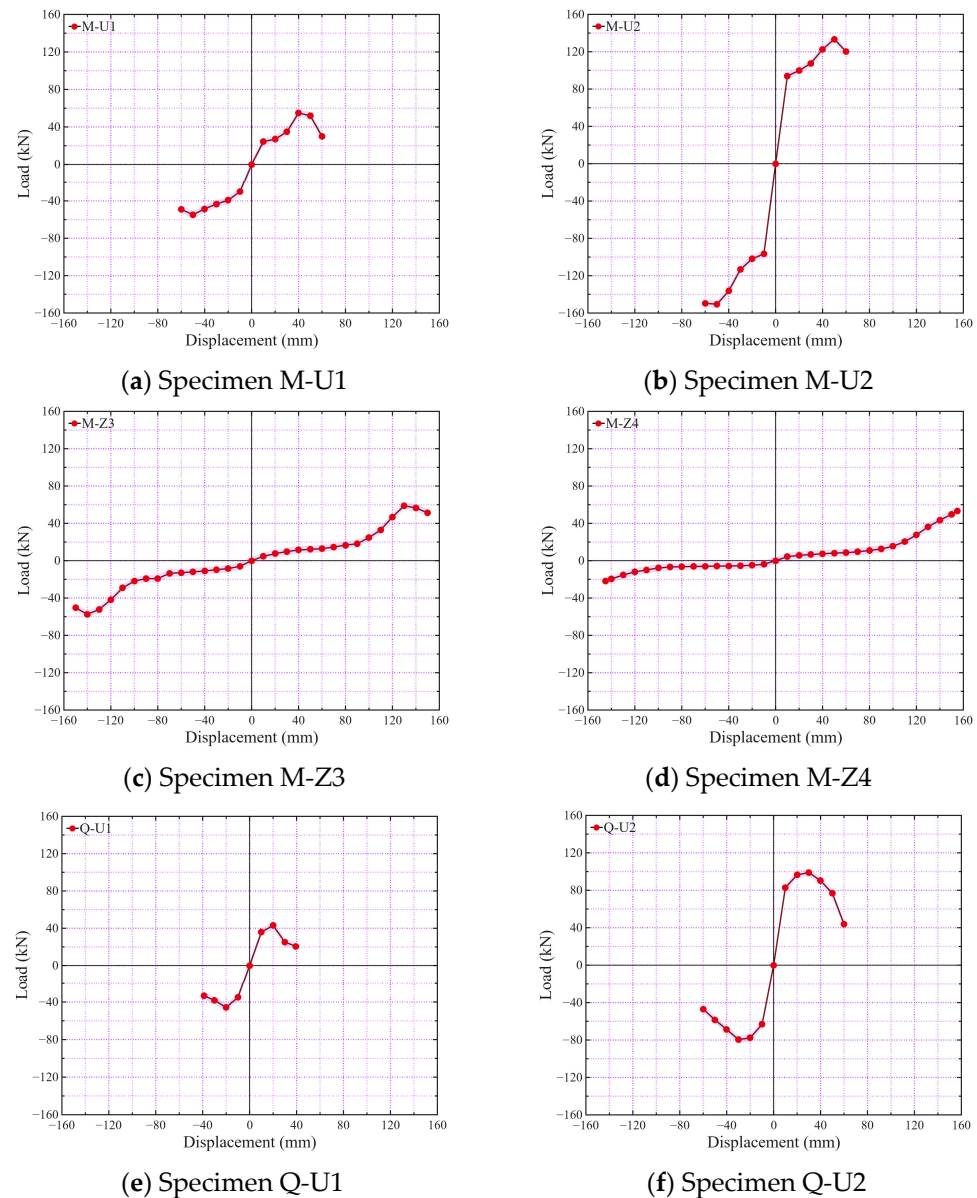


Figure 8. Skeleton curves, showing rapid post-peak degradation for the U-type walls and stable resistance of the Z-type walls within the tested displacement range.

Table 4. Peak and test-end response parameters of specimens.

Specimen	μ	Δ_y (mm)	P_y (kN)	Δ_{max} (mm)	P_{max} (kN)	Δ_e (mm)	P_e (kN)	θ_e (%)	P_e/P_{max}
M-U1	2.33	22.91	26.82	44.96	54.98	53.37	46.73	1.80	0.85
M-U2	4.36	13.82	91.40	50.56	132.19	60.27	112.36	1.47	0.85
M-Z3	—	116.8	41.81	134.96	63.07	148.34	53.61	5.00	0.85
M-Z4	—	130.74	40.18	155.37	53.57	155.37	53.57	3.79	1.00
Q-U1	2.15	11.92	36.20	19.93	42.15	25.63	35.83	0.86	0.85
Q-U2	4.91	10.00	82.70	32.06	97.94	49.08	83.25	1.20	0.85

Note: Δ_e and P_e denote the displacement and lateral load at test termination, respectively. For specimens M-U1, M-U2, Q-U1, and Q-U2, the prescribed structural failure criterion was reached; therefore, $\Delta_e = \Delta_u$ and $P_e = P_u$. For specimens M-Z3 and M-Z4, the tests were terminated because the actuator reached its displacement range limit before structural failure occurred; therefore, $\Delta_e = \Delta_t$ and $P_e = P_t$, representing the maximum tested displacement and corresponding terminal load. The conventional displacement ductility coefficient was not determined for the two Z-type specimens. θ_e is the inter-story drift ratio at test termination.

The Q-U specimens were sensitive to stress concentration because of the relatively low stiffness and resistance of the SFC panels. Pronounced asymmetry was observed, particularly for specimen Q-U2. After the peak load, local crushing and rapid crack propagation caused a marked decrease in lateral resistance. The calculated displacement ductility coefficients ranged from 2.15 to 4.91. However, these values should not be interpreted independently as evidence of favorable ductility because local damage and apparent yielding occurred at relatively small displacements, while the post-peak energy-dissipation and strength-retention capacities remained limited.

For the M-Z series, the load-transfer responses in the positive and negative directions were generally consistent, and only minor differences were observed between the positive and negative peak loads. Owing to the slip mechanism of the Z-type connections, the initial slope of the skeleton curves was relatively low and the apparent yielding point was delayed. Because neither M-Z3 nor M-Z4 reached the prescribed failure criterion, their true ultimate displacements and conventional displacement ductility coefficients could not be determined. Therefore, the relatively low ratios obtained by dividing the terminal displacement by the apparent yield displacement should not be interpreted as ultimate ductility coefficients. Instead, the deformation performance of the Z-type specimens was evaluated using the maximum tested inter-story drift ratio, load retention, residual stiffness, cumulative energy dissipation, and observed damage. Up to the maximum tested displacements, the specimens maintained stable load-carrying behavior, and only localized corner cracking was observed.

Under the tested U-type connection condition, the measured peak loads of the MRC specimens were approximately 30–35% higher than those of the corresponding SFC specimens, while their ultimate displacements were approximately 1.2–2.1 times those of the SFC specimens. Compared with the brittle local crushing observed in the SFC specimens, the MRC specimens exhibited a semi-brittle response with greater deformation tolerance. For the tested MRC walls, the apparent yield displacements of the Z-type specimens were approximately 5.1–9.5 times those of the corresponding U-type specimens, while their maximum tested displacements were approximately 2.58–2.78 times the ultimate displacements of the U-type specimens.

3.2.3. Stiffness Degradation Law

Under low-cycle reversed loading, stiffness degradation provides an important indication of the progressive damage accumulated in a structural system. However, stiffness degradation alone cannot fully characterize the seismic performance of wall systems. Previous research on coupled and hybrid-coupled shear walls has similarly evaluated cyclic behavior by jointly considering damage distribution, secant stiffness, ductility, and energy dissipation [5]. Although the structural configurations differ, the present study adopts the same multi-index perspective by interpreting stiffness degradation together with failure modes, deformation capacity, residual resistance, and cumulative energy dissipation. The secant stiffness was calculated using Equations (2) and (3):

$$K_i = \frac{|+P_i| + |-P_i|}{|+X_i| + |-X_i|} \quad (2)$$

$$\delta_K = \frac{K_0 - K_{\text{res}}}{K_0} \times 100\% \quad (3)$$

In Equations (2) and (3), $+P_i$ and $-P_i$ are the positive and negative peak loads at the i -th loading stage. $+X_i$ and $-X_i$ are the corresponding peak displacements. K_0 is the initial stiffness. K_{res} is the residual stiffness. δ_K is the stiffness degradation rate. The

stiffness degradation curves of the wall specimens are shown in Figure 9. The core stiffness parameters are listed in Table 5.

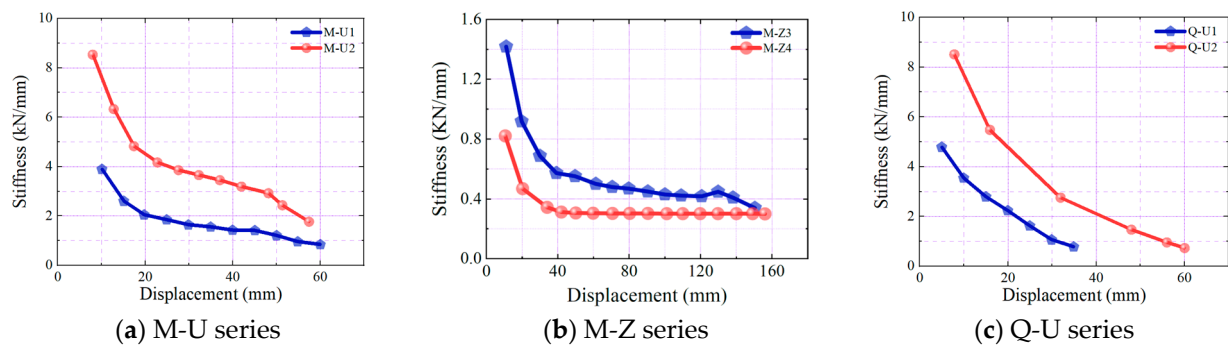


Figure 9. Stiffness degradation curves, showing rapid stiffness loss in the U-type specimens and a gradual slip-dominated response in the Z-type specimens.

Table 5. Initial stiffness, residual stiffness, and stiffness degradation rate of the specimens.

Specimen	K_0 (kN/mm)	K_{res} (kN/mm)	δ_K (%)
M-U1	3.89	0.84	78.41
M-U2	8.53	1.77	79.25
M-Z3	1.41	0.34	75.89
M-Z4	0.82	0.30	63.41
Q-U1	4.77	0.73	84.70
Q-U2	8.47	0.78	90.79

Due to the absence of deformation buffering, the stiffness of the U-type joints degraded rapidly. Under rigid constraint conditions, stress concentrations were easily induced in the straw fiber cement panels. Consequently, the stiffness degradation rate reached 84.7% to 90.8%. With residual stiffness dropping to only 9% to 15% of the initial value, a clear brittle failure mode was demonstrated. By contrast, a more gradual damage process was observed for the MRC wall panels. For these specimens, the degradation rate ranged from 78.4% to 79.3%, which was 6% to 12% lower than that of the straw panels. Furthermore, the residual stiffness was maintained at approximately 21% of the initial value.

For the Z-type joints, segmented stiffness degradation characteristics were clearly exhibited. Dominated by the slip mechanism, the initial stiffness remained low, and the early degradation was slow. Once the loading displacement reached the range of 120 to 140 mm, a slight stiffness hardening was induced by the tightening of the boundary constraints. Even at a large displacement of 155 mm, a residual stiffness of 0.30 to 0.34 kN/mm was retained by the specimens. This behavior effectively prevented a sudden decrease in the load-carrying capacity.

Based on the measured load–displacement response, displacement-monitoring results, and stiffness-degradation curves, the force–deformation evolution of the Z-type joints can be divided into three successive stages. During the initial free-slip stage, the bolts moved within the reserved clearances of the slotted holes, while the Z-shaped connectors underwent relative slip and rotation. The wall panels were not fully engaged in lateral resistance, resulting in low initial stiffness and pronounced slip plateaus in the hysteretic curves. During the contact load-transfer stage, the bolts gradually contacted the ends of the slotted holes. Friction at the contact interfaces, bearing between the bolts and slot edges, and bending deformation of the Z-shaped steel plates progressively activated the load-transfer path from the steel frame to the wall panels. During the boundary-tightening stage, the available slip was nearly exhausted and the restraint provided by the

connections gradually increased. The wall panels then participated more fully in resisting the lateral load, resulting in the slight stiffness strengthening observed at displacements of approximately 120–140 mm. Therefore, the stiffness evolution of the Z-type joints originated from the sequential transition from free slip to contact bearing and finally to strengthened boundary restraint.

Interestingly, a longer slip plateau was exhibited by specimen M-Z4 compared with M-Z3. This phenomenon is primarily attributed to the structural characteristics of the Z-type joints and the influence of installation tolerances. Because of the inherent reserved gaps in the Z-type joints, the lateral resistance was very low during the initial loading stage, forming a clear pinching effect. Particularly for large-scale components like M-Z4, initial installation tolerances were magnified due to difficulties in hoisting and alignment. Therefore, the reserved gap is larger than that of specimen M-Z3. Driven by these enlarged gaps, the components had to undergo a longer free-slip distance to achieve physical engagement before bearing loads. This mechanical requirement resulted in the extended initial slip section observed in M-Z4.

Regarding the U-type connections, the initial stiffness was increased by enlarging the wall dimensions. Specifically, the initial stiffness of the large-scale straw and MRC panels increased by 78% and 119%, respectively. This increase effectively narrowed the stiffness gap between the two materials during the initial micro-deformation stage. However, for the Z-type connections, the rigid constraints exerted by the frame on the wall panels were dissipated by the joint slip. Thus, an increase in geometric dimensions did not yield a proportional increase in the initial stiffness. These findings indicate that the deformation of large-scale components can be effectively coordinated by the flexible joints through slip mechanisms. Ultimately, the influence of the wall panel dimensions on the global structural performance was mitigated.

3.2.4. Energy Dissipation Performance

The energy dissipation curves are shown in Figure 10. The energy dissipation parameters are listed in Table 6. The differences in energy dissipation performance are analyzed based on the working mechanisms of the two connections. The energy dissipation coefficient η is calculated by Equation (4). The equivalent viscous damping coefficient ζ_{eq} is calculated by Equation (5):

$$\eta = \frac{S_{(ABC+CDA)}}{S_{(OBE+ODF)}} = 2\pi \cdot \zeta_{eq} \quad (4)$$

$$\zeta_{eq} = \frac{1}{2\pi} \cdot \frac{S_{(ABC+CDA)}}{S_{(OBE+ODF)}} \quad (5)$$

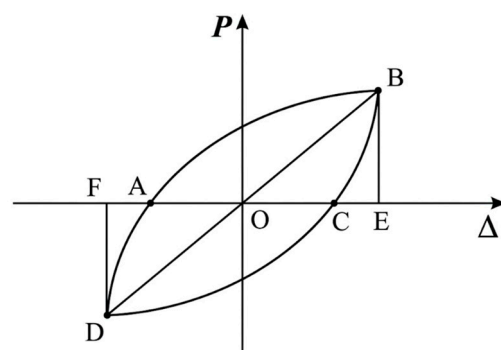


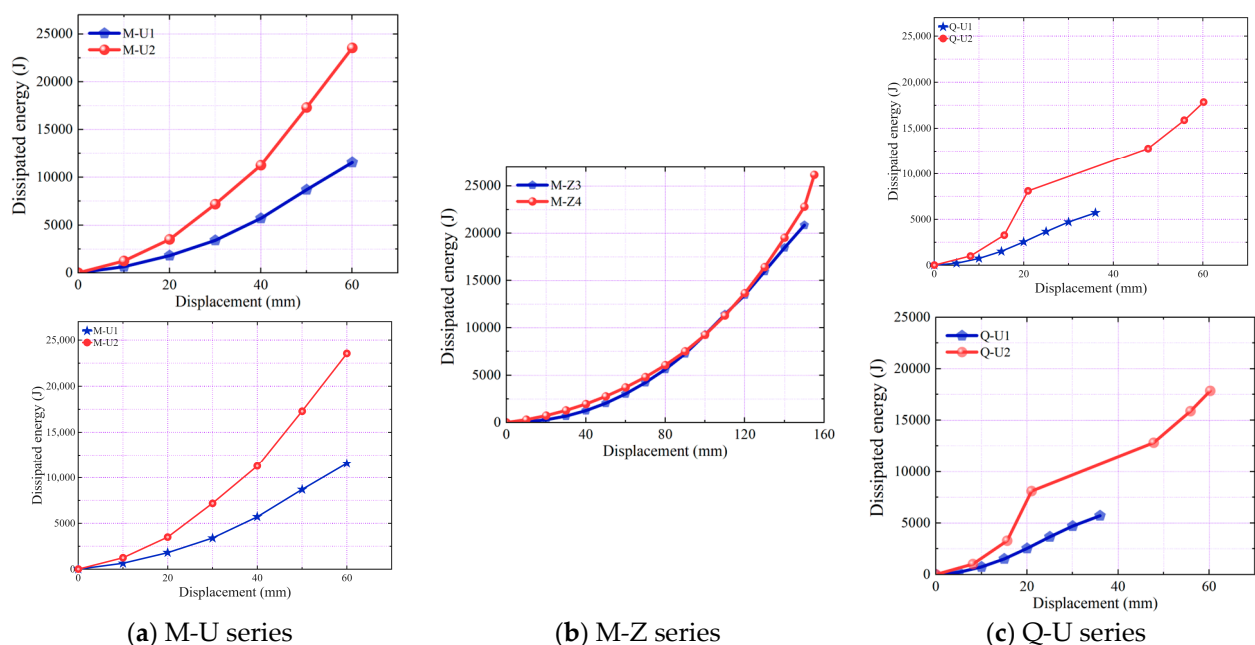
Figure 10. Equivalent viscous damping coefficients, with higher per-cycle values for the rigidly connected specimens.

Table 6. Energy-dissipation coefficients, equivalent viscous damping ratios, and cumulative dissipated energy.

Specimen	H	ζ_{eq}	$E(J)$
M-U1	1.13	0.18	11,561.03
M-U2	2.14	0.34	23,544.79
M-Z3	0.97	0.15	20,842.25
M-Z4	1.07	0.17	26,165.17
Q-U1	0.82	0.13	5706.26
Q-U2	1.28	0.20	17,846.97

In Equations (4) and (5), $S_{(ABC+CDA)}$ is the area enclosed by the hysteresis loop. η is the energy dissipation coefficient. ζ_{eq} is the equivalent viscous damping coefficient.

For the U-type joint system, energy dissipation was primarily dominated by the plastic damage of the wall panels. During the initial loading stage, the superior mechanical properties of the MRC material provided the M-U series with a high single-cycle energy dissipation capacity. Specifically, the η of specimen M-U2 reached 2.14, and the corresponding ζ_{eq} was 0.34. However, while a high initial stiffness was provided by the rigid U-type joints, this constraint also led to the premature exhaustion of energy dissipation capacity in the later stages. As illustrated in Figure 11a, loading of the specimens was terminated at a displacement of approximately 60 mm. With increasing loading displacement, sequential severe damage occurred in the wall panels. Consequently, the large-deformation capacity of the system was limited after the peak load was exceeded. By contrast, the Q series was more prone to crushing due to the higher porosity of the material. As shown in Figure 11c, specimen Q-U1 failed at a very small displacement, demonstrating a poor capacity for large-deformation coordination. Ultimately, the total cumulative energy dissipation of this specimen was only 5706.26 J, which was generally low.

**Figure 11.** Cumulative energy dissipation curves, showing continued energy accumulation in the Z-type specimens over a larger displacement range.

For the Z-type joint system, the energy dissipation mechanism shifted to a synergistic mode consisting of friction slip and localized plasticity. Although the energy dissipation coefficient η was limited to the range of 0.97 to 1.07, the risk of premature crushing in

the MRC wall panels was successfully avoided due to the slip mechanism of the Z-type joints. By implementing this slip mechanism, the specimens maintained a stable cyclic response up to a maximum tested displacement of approximately 155 mm. Consequently, cumulative energy dissipation continued to increase throughout the tested displacement range. The cumulative energy dissipation values of M-Z3 and M-Z4 reached 20.84 and 26.17 kJ, respectively. At test termination, their load-retention ratios were approximately 0.85 and 1.00, respectively, confirming that no abrupt strength loss occurred before the actuator displacement limit was reached.

3.2.5. Synergistic Effect of Connection Type and Wall Panel Material

The seismic response of the composite walls was governed by a sequential interaction between connection deformation and wall panel resistance. Under the U-type rigid connections, the frame displacement was directly transferred to the wall panels, resulting in early stress concentration and cracking. Under this connection condition, the influence of the panel material was pronounced. Compared with the SFC specimens, the MRC specimens exhibited 30–35% higher ultimate bearing capacity and slower stiffness degradation, indicating greater strength and damage tolerance.

For the Z-type flexible connections, the slip and rotation of the slotted holes initially released the frame deformation and delayed the substantial engagement of the wall panels. After the available slip was gradually exhausted, the MRC panels began to participate more significantly in the lateral resistance and maintained the load-carrying capacity at large displacements. Consequently, cracking was delayed to 130–155 mm, and stable resistance was maintained up to displacements of approximately 148–155 mm. These results indicate that the connection type controls when and how the wall panel participates in load transfer, whereas the panel material controls the resistance and damage tolerance after engagement. This complementary load-transfer sequence constitutes the synergistic mechanism between the dry connection and the wall panel. However, because Z-type specimens with SFC panels were not included in the present tests, the identified interaction mechanism is limited to the tested configurations and requires further verification.

3.3. Theoretical Analysis Based on Continuum Damage Mechanics (CDM)

3.3.1. Basic Restoring Force Model in Continuum Mechanics

Based on Lemaitre's strain energy equivalence hypothesis, the true complementary strain energy of the wall with micro-defects equals the strain energy of the undamaged material under effective stress. The macroscopic energy expression is defined by Equation (6):

$$W_e = \frac{1}{2} \cdot P_i \cdot \Delta_i = \frac{1}{2} \cdot [K_0 \cdot (1 - D_i)] \cdot \Delta_i^2 \quad (6)$$

The partial derivative of the energy equation is taken with respect to displacement. This yields the nonlinear secant stiffness restoring force (Equation (7)):

$$P_i = \frac{\partial W_e}{\partial \Delta_i} = K_0 \cdot (1 - D_i) \cdot \Delta_i \quad (7)$$

Through further simplification, Equation (8) is obtained:

$$P_i = K_0 \cdot \Delta_i \cdot (1 - D_i) \quad (8)$$

In Equation (8), P_i is the macroscopic restoring force at any loading state (kN). Δ_i is the loading displacement at any time (mm). D_i is the comprehensive damage variable at any state.

During the initial loading stage, the composite damage variable D_i approaches zero. Consequently, the lateral stiffness of the structure is entirely governed by the initial stiffness K_0 . As indicated by the data in Table 7, premature load-bearing in the wall panels was induced by the rigid deformation compatibility of the U-type connections. This early engagement resulted in a relatively high initial stiffness. Specifically, when the wall height increased from 2966 mm to 4100 mm, the K_0 of the M-U series increased from 3.89 to 8.53 kN/mm. In contrast, deformation constraints were effectively weakened by the Z-type connections through the slip of the slotted holes. Simultaneously, early energy was dissipated by relying on joint friction. Driven by this mechanism, the K_0 of these specimens was maintained at a low level, ranging from 0.82 to 1.41 kN/mm.

Table 7. Measured specimen initial lateral stiffness.

Specimen	H (mm)	Connection Type	K_0 (kN/mm)
M-U1	2966	U-type	3.89
M-U2	4100	U-type	8.53
M-Z3	2966	Z-type	1.41
M-Z4	4100	Z-type	0.82
Q-U1	2966	U-type	4.77
Q-U2	4100	U-type	8.47

3.3.2. Peak-State Damage Variable

To quantify the true internal damage of the wall panels under the experimental peak-load state, the peak-load point is extracted. Guided by the strain equivalence hypothesis of Lemaitre's continuum damage mechanics, the peak damage variable corresponding to this state is defined as D_p . Based on the definition of the secant stiffness K_p , this relationship is formulated as Equation (9):

$$K_p = \frac{P_{\max}}{D_{\max}} \quad (9)$$

By combining the CDM restoring-force relationship in Equation (8) with the definition of K_p , the peak-state damage variable is obtained, as formulated in Equation (10):

$$D_p = 1 - \frac{K_p}{K_0} = 1 - \frac{P_{\max}}{K_0 \cdot \Delta_{\max}} \quad (10)$$

As indicated by the calculation results summarized in Table 8, deformations could not be released by the U-type specimens due to the rigid constraints. Consequently, energy dissipation was highly dependent on the damage of the wall panels. This mechanism resulted in peak damage variables D_p ranging from 0.56 to 0.69. Thus, a significant risk of brittle failure was posed to the structural system. In contrast, the slip and rotation mechanisms of the Z-type connectors enabled the specimens to accommodate substantially larger global displacements. Driven by this extended friction-induced work, the peak secant stiffness K_p decreased to the range of 0.34 to 0.47 kN/mm. Simultaneously, energy was effectively buffered, and the wall panels were protected. Within the tested displacement range, premature global failure of the walls was not observed, and stable load-carrying behavior was maintained. Because the Z-type specimens did not reach structural failure, these results should not be interpreted as measures of ultimate ductility.

Table 8. Peak-State Damage Variables of the Tested Specimens.

Specimen	$P_{\max}$ (kN)	$\Delta_{\max}$ (mm)	Δ_u (mm)	K_p (kN/mm)	D_p
M-U1	54.98	44.96	53.37	1.22	0.69
M-U2	132.19	50.56	60.27	2.61	0.69
M-Z3	63.07	134.96	148.34	0.47	0.67
M-Z4	53.57	155.37	155.37	0.34	0.58
Q-U1	42.15	19.93	25.63	2.11	0.56
Q-U2	97.94	32.06	49.08	3.05	0.64

3.3.3. Peak-Load Restoring-Force Verification

To evaluate the predictive capability of the proposed model for the tested configurations, the average theoretical peak damage variable $\bar{D}_p^-$ of each specimen series was extracted. This calculated average was defined as the characteristic damage parameter for the corresponding wall configuration. Subsequently, the measured displacement at the peak load $\Delta_{\max}$ and the average peak damage variable $\bar{D}_p^-$ were substituted into the macroscopic restoring-force relationship in Equation (8), as formulated in Equation (11):

$$P_{\text{cal}} = K_0 \cdot (1 - \bar{D}_p^-) \cdot \Delta_{\max} \quad (11)$$

This result is compared with the experimental peak load $P_{\max}$. The error rate δ is defined by Equation (12):

$$\delta = \left| \frac{P_{\text{cal}} - P_{\max}}{P_{\max}} \right| \times 100\% \quad (12)$$

As verified by the data in Table 9, The theoretically calculated peak loads were generally consistent with the experimental peak loads of the tested specimens. The relative errors were within 1.5% for the M-U series, approximately 10% for the Q-U series, and 9.63–13.14% for the M-Z series. These results indicate that the model provides reasonable estimates for the configurations considered in this study. Its applicability to other panel materials, connection details, axial load levels, and wall configurations requires further validation.

Table 9. Comparison between Theoretical and Experimental Peak Loads.

Specimen	$\bar{D}_p^-$	P_{cal} (kN)	$P_{\max}$ (kN)	δ (%)
M-U1	0.69	54.21	54.98	1.40
M-U2		133.70	132.19	1.14
M-Z3	0.62	71.36	63.07	13.14
M-Z4		48.41	53.57	9.63
Q-U1	0.60	38.02	42.15	9.78
Q-U2		108.61	97.94	10.90

4. Finite Element (FE) Numerical Simulation and Verification

4.1. Establishment of Finite Element Model

Three-dimensional solid models were established in ANSYS Workbench 2002R2. The eight-node SOLID65 element was used to model the wall panels. The element formulation is capable of representing tensile cracking and compressive crushing. The MRC concrete strength grade was set to C20, and the shear-transfer coefficients for open and closed cracks were taken as 0.50 and 0.95, respectively. To improve nonlinear convergence, the concrete-crushing option was disabled in the present analysis. Consequently, the model was used to reproduce the global response, cracking-related stiffness redistribution, and stress concentration regions, but it could not explicitly simulate local crushing, material fragmentation, or the complete post-peak degradation process.

The boundary conditions were defined according to the test setup. The bottom steel frame was fully fixed to simulate its anchorage to the laboratory floor. A monotonically increasing horizontal displacement was applied at the top of the frame. The monotonic analysis was adopted to reproduce the envelope response represented by the experimental skeleton curves and to identify the principal load-transfer paths and stress concentration regions. It was not intended to reproduce the complete cyclic response, including unloading and reloading paths, hysteretic pinching, cycle-dependent stiffness degradation, cumulative damage, and hysteretic energy dissipation. Different contact types were assigned to the interfaces according to their mechanical behavior. Bonded contact was used at the intersections of the steel frame members to ensure reliable force transfer. Frictional contact was defined between adjacent wall panels and between the wall panels and the surrounding steel members. The effective friction coefficients were taken as 0.20 for the panel-to-panel interfaces and 0.10 for the panel-to-steel interfaces. These values were selected with reference to friction parameters adopted in previous finite element studies of concrete–steel and component contact interfaces [45,46] and were not obtained from direct interface tests. Therefore, they should be regarded as effective modeling parameters. The actual interface response may also be influenced by surface roughness, contact pressure, construction tolerance, and installation gaps. Bonded contact was assigned between the wall panels and the bottom steel beam to prevent relative slip and to maintain stable load transfer.

A combined meshing strategy of hexahedral mapping and hex-dominant adaptive meshing was adopted. The mesh size of the SOLID65 elements was set to 60 mm. This size provided a balance between computational efficiency and numerical accuracy. After meshing, the mesh smoothness was set to medium and the transition type was defined as slow. The final finite element model is shown in Figure 12. The average mesh quality was 0.92, indicating good mesh quality.

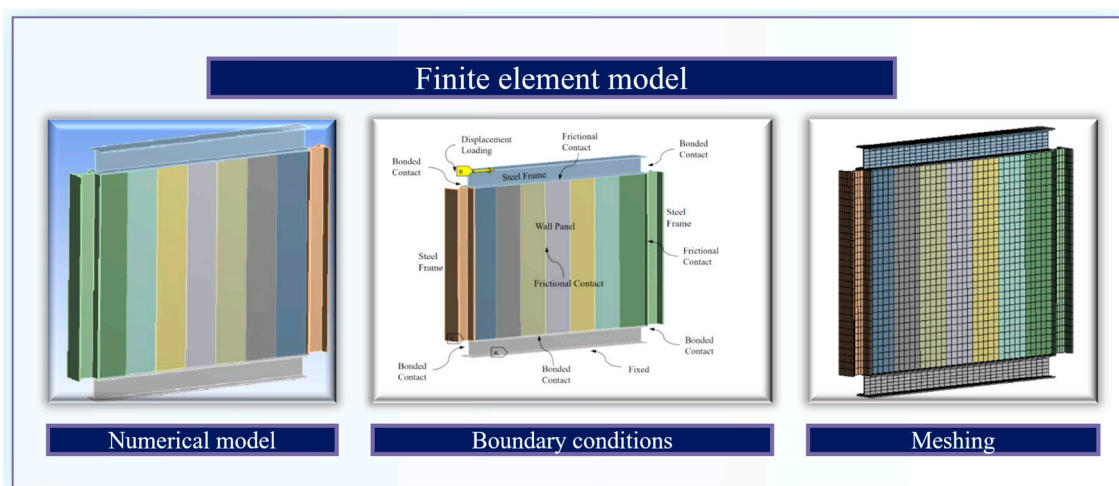


Figure 12. Finite Element Model.

4.2. Constitutive Model of Materials

4.2.1. Steel Constitutive Model

Regarding the steel constitutive properties, an ideal elastoplastic model was adopted for the steel beams, steel columns, and connecting components. The corresponding material constitutive relationship is illustrated in Figure 13. Within this model, a linearly monotonic increase is exhibited by the stress–strain relationship prior to material yielding. Following the yield point, the stress–strain behavior transitions into a horizontal line. Furthermore,

the elastic modulus of the material is identical in tension and compression. This specific constitutive relationship is mathematically expressed by Equation (13):

$$\sigma = \begin{cases} E_s \varepsilon, & 0 \leq \varepsilon \leq \varepsilon_y \\ f_y, & \varepsilon_y \leq \varepsilon \leq \varepsilon_u \end{cases} \quad (13)$$

In Equation (13), E_s is the elastic modulus of steel. f_y is the yield strength of steel. ε_y is the yield strain. ε_u is the ultimate strain.

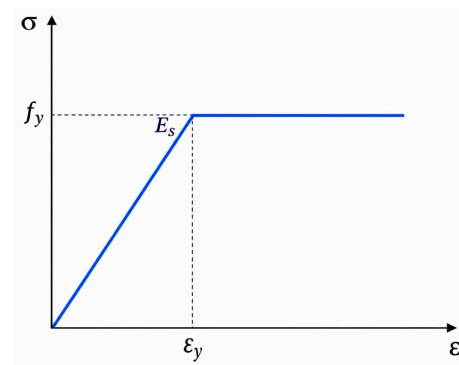


Figure 13. Steel constitutive relationship.

4.2.2. Concrete Constitutive Model

The concrete in the MRC and straw composite wall panels was modeled in ANSYS using the SOLID65 element, which is capable of simulating concrete cracking and crushing behavior. The uniaxial tensile constitutive relationship was defined in accordance with the provisions of GB 50010-2010 [35]. Damage parameters were introduced to account for the progressive degradation of material stiffness during cracking and inelastic deformation.

The measured elastic moduli and compressive strengths of the two wall-panel materials were directly adopted in the finite element model. The Poisson's ratio was taken as 0.20 for both materials. The tensile response after cracking was represented by a linear softening law based on the fracture-energy approach. For the MRC panels, the tensile cracking strength and fracture energy were taken as 2.00 MPa and 0.10 N/mm, respectively. For the SFC panels, the corresponding values were 0.63 MPa and 0.05 N/mm. The crack opening displacement at zero tensile stress was determined as $w_c = 2G_f / f_t$, giving values of 0.100 mm and 0.159 mm for the MRC and SFC panels, respectively. The shear-transfer coefficients for open and closed cracks were set to 0.50 and 0.95, respectively. An element size of 60 mm was adopted, and the concrete crushing option was disabled to improve nonlinear convergence. In the SOLID65 smeared-crack model, stiffness degradation was represented implicitly through crack formation, tensile softening, and reduced shear transfer across the crack surfaces; therefore, an independent scalar damage factor was not specified. The principal material and numerical parameters are summarized in Table 10.

Table 10. Material and numerical parameters used in the wall-panel models.

Parameter	MRC	SFC
Elastic modulus E (MPa)	22,000	814.5
Poisson's ratio ν	0.20	0.20
Compressive strength f_c (MPa)	19.96	6.30
Tensile cracking strength f_t (MPa)	2.00	0.63
Fracture energy G_f (N/mm)	0.10	0.05
Open/closed crack coefficient	0.50/0.95	0.50/0.95
Mesh size (mm)	60	60

Determining the concrete stress–strain curve is the first step in the nonlinear finite element analysis. This relationship forms the basis for studying concrete strength, cracking, and deformation. In this study, the concrete constitutive relationships are cited from the literature [35]. The uniaxial tension and compression stress–strain curves are shown in Figure 14. The mathematical expressions are determined by the following equations:

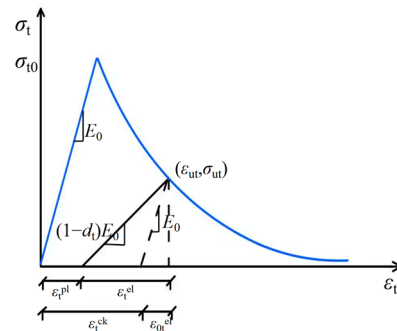


Figure 14. Concrete constitutive relationship.

4.3. Model Test Verification

4.3.1. Validation of Failure Modes

The simulated stress contours are consistent with the experimental failure modes, as shown in Figure 15. The numerical results reveal the stress distribution and transfer path in the wall panels during loading. Stress concentration occurs mainly at the panel corners and vertical splicing joints. These regions correspond well to the locations where cracks initiated and propagated in the tests. The finite element model reproduced the principal stress concentration regions and the cracking-related response of the wall panels. High tensile and compressive stresses were mainly distributed at the panel corners and vertical joints, corresponding to the experimentally observed crack-initiation and crushing-prone regions. Because the concrete-crushing option was disabled, local crushing and material fragmentation were not explicitly simulated. Therefore, the numerical results are used to identify the stress conditions associated with the observed damage locations rather than to reproduce the complete physical damage process. Therefore, the numerical results provide a basis for interpreting the stress distribution and damage-prone regions of the tested configurations.

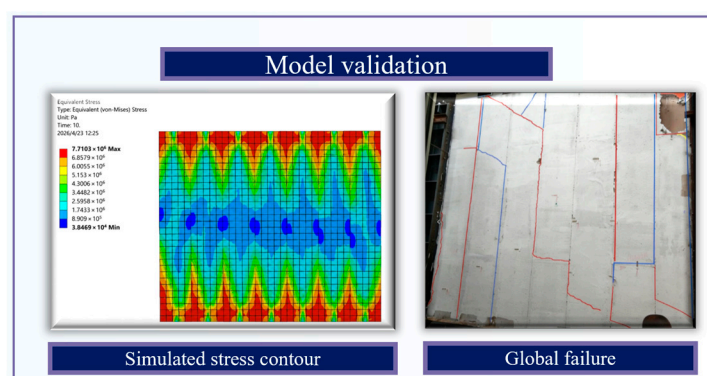


Figure 15. Comparison of experimental damage and simulated stress-concentration regions for representative specimens.

4.3.2. Comparison of Skeleton Curves

To avoid the excessive computational cost and cumulative numerical errors associated with low-cycle reversed loading, monotonic horizontal loading was adopted to obtain the

skeleton curves, which were subsequently compared with the experimental results. As shown in Figure 16a, for the U-type connection, the simulated and experimental curves were in close agreement. During the initial loading stage, the two curves exhibited highly similar responses, and the structure rapidly reached the yield load at a very small displacement, followed by a stable strengthening stage. The numerical model accurately reproduced the positive peak load of approximately 133 kN, as well as the post-peak descending branch, thereby capturing the rigid load-transfer characteristics of the U-type connection.

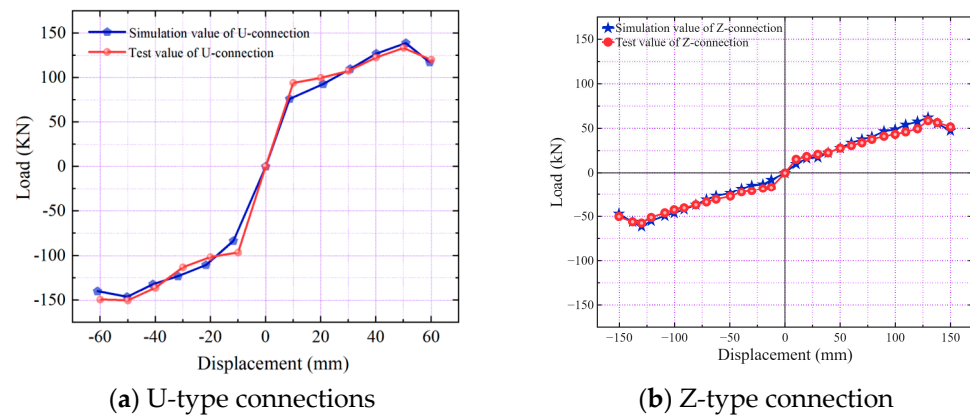


Figure 16. Comparison of experimental and simulated skeleton curves, showing close agreement in the overall envelope trends.

As shown in Figure 16b, for the simulation of the Z-series specimens, the initial displacement tolerance was predefined in the contact pairs according to the average boundary gap measured in the tests. The numerical model reasonably reproduced the general envelope response of the Z-type connection despite its relatively complex force-transfer mechanism. Under positive loading, the predicted peak load was 65.85 kN, compared with the experimentally recorded value of 63.07 kN, corresponding to a relative difference of 4.41%. The model captured the extended sliding stage during the early and intermediate loading stages, together with the subsequent increase in lateral resistance at large displacements. Because the test was terminated by the actuator displacement limit before structural failure occurred, this comparison should not be interpreted as validation of the true ultimate or failure response of the Z-type connection.

Overall, the numerical models showed reasonable agreement with the experimental skeleton curves in terms of peak resistance and general envelope-response trends. The comparison supports their use for interpreting the load-transfer mechanisms of the tested configurations within the adopted assumptions.

4.3.3. Comparison of Secant Stiffness Evolution

Figure 17 compares the secant stiffness calculated from the monotonic numerical response with that obtained from the envelope points of the experimental cyclic response. The comparison is intended to evaluate the global stiffness variation along the skeleton curve rather than cycle-dependent stiffness degradation. As shown in Figure 17a, both the simulated and experimental curves for the U-type connection exhibited a characteristic trend of rapid degradation in the initial stage followed by a gradual stabilization at later stages. At the initial displacement point, the measured and simulated stiffness values were 8.53 kN/mm and 7.97 kN/mm, respectively, corresponding to a relative error of approximately 6.57%. By the end of loading, the stiffness had degraded to 1.77 kN/mm in the experiment and 1.95 kN/mm in the simulation, accurately reflecting the stiffness degradation behavior of the U-type connection. The agreement in the overall secant-stiffness trend supports the use of the model for interpreting the envelope response of

the tested specimens. It should not be interpreted as direct validation of cyclic damage accumulation or repeated-loading stiffness degradation.

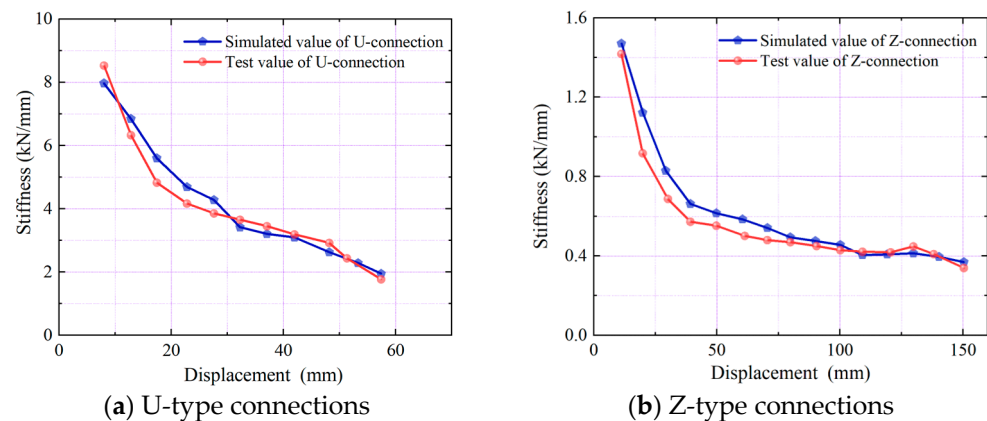


Figure 17. The stiffness evolution.

As shown in Figure 17b, the numerical and experimental secant-stiffness curves of the Z-type connection exhibited similar overall trends over the displacement range of 10–150 mm. At the initial point, the difference between the simulated and measured stiffness values was approximately 3.67%. Subsequently, as the sliding mechanism of the connection was progressively activated, the secant stiffness decreased and entered an extended low-stiffness stage. At the end of loading, the simulated and experimental stiffness values were 0.37 kN/mm and 0.34 kN/mm, respectively. The agreement at the characteristic points and in the overall secant-stiffness trend supports the use of the monotonic model for interpreting the envelope response of the tested specimen; however, it does not constitute direct validation of cyclic stiffness degradation or damage evolution.

4.4. Limitations of the Present Study

Several limitations of the present study should be acknowledged. First, the experimental program included six full-scale specimens, with only one specimen for each configuration. Therefore, specimen-to-specimen variability and statistical significance could not be quantified. Second, the test matrix was not a complete factorial design because SFC panels were tested only with U-type connections, whereas Z-type connections were tested only with MRC panels. Accordingly, the interaction between connection type and wall panel material cannot be completely separated from the current experimental results. The finite element analyses provide additional information on the response trends and load-transfer mechanisms but do not replace the missing experimental combinations or repeated tests. Future work should include SFC walls with Z-type connections, repeated specimens, and additional variations in axial load level, connection dimensions, wall configurations, and boundary conditions.

In addition, the environmental performance of the MRC and SFC panels was not quantitatively evaluated in the present study. No life-cycle assessment or embodied-carbon analysis was conducted because complete inventory data concerning raw-material production, manufacturing, transportation, construction, maintenance, service life, and end-of-life treatment were unavailable. Therefore, the present results do not establish the environmental superiority of the investigated panels over conventional wall systems. Future research should conduct a life-cycle assessment based on complete inventory data and compare the embodied carbon of the investigated panels with that of conventional wall systems.

The finite element model also involved several simplifying assumptions. Monotonically increasing horizontal displacement was adopted to evaluate the envelope response represented by the experimental skeleton curves rather than the complete cyclic hysteretic behavior. Therefore, unloading and reloading paths, pinching behavior, cycle-dependent stiffness degradation, cumulative damage, and hysteretic energy dissipation were not directly simulated. In addition, the concrete-crushing option was disabled to improve nonlinear convergence. Consequently, the model could identify compressive stress concentration and crushing-prone regions but could not explicitly reproduce local crushing, material fragmentation, or the complete post-peak degradation process.

The friction coefficients for the panel-to-panel and panel-to-steel interfaces were selected with reference to values and ranges reported in previous numerical studies and were treated as effective modeling parameters rather than directly measured material constants. The actual interface behavior may also be affected by surface roughness, contact pressure, construction tolerance, bolt pretension, and installation gaps. Because no dedicated interface tests were conducted, the quantitative sensitivity of the numerical response to the friction coefficients, contact stiffness, bolt pretension, and installation gaps was not independently evaluated. Accordingly, the numerical results are used primarily to interpret the global envelope response, principal stress distribution, and load-transfer mechanisms of the tested configurations. Future work will include direct interface characterization, cyclic numerical modeling, and systematic parameter sensitivity analyses to further calibrate and validate the numerical model.

5. Discussion

5.1. Comparison with Previously Published Dry-Connected Wall Systems

Previous dry-connected wall systems have employed bolted plates, U-shaped channels, Z-shaped steel connectors, and other steel joints to improve assembly efficiency and transfer lateral loads [5,12–15,33]. These studies mainly emphasize connector strength, joint capacity, deformation capacity, or post-earthquake recoverability. In contrast, the present Z-type connection uses a slotted hole to permit in-plane slip and rotation. Within the tested MRC specimens, visible cracking was delayed from 8–10 mm for the U-type connection to 130–155 mm for the Z-type connection, and the tested displacement range increased from approximately 53–60 mm to 148–155 mm. These observations indicate an advantage in deformation accommodation and panel-damage control.

However, the Z-type system also exhibited lower initial stiffness and more pronounced hysteretic pinching than the U-type system. A direct quantitative ranking against previously published systems is not appropriate because the panel materials, axial-load levels, wall dimensions, connector details, loading protocols, and failure criteria differ among studies. In addition, connector replaceability, residual deformation, post-earthquake re-centering, and performance under out-of-plane, fire, and durability demands were not evaluated. Therefore, the relative advantages and limitations discussed here apply only to the tested configurations.

5.2. Experimental Evidence and Analytical Interpretation

The directly observed experimental evidence includes the cracking displacements, damage patterns, hysteretic and skeleton curves, stiffness degradation, and dissipated energy reported in Section 3. The proposed mechanism that slotted-hole slip and rotation redistribute deformation away from the panels is an interpretation supported by the slip plateaus and the FE stress distributions. Because local connector slip, bolt force, interface friction, and panel strains were not measured independently, the individual contributions of these mechanisms cannot be quantified from the present tests.

The CDM and FE analyses should therefore be regarded as interpretive tools rather than independent experimental evidence. The CDM damage variable is derived from global stiffness and does not directly measure material microdamage. Similarly, the FE model identifies stress-concentration regions and envelope-response trends, but it does not directly verify every local damage mechanism or the complete cyclic response. These distinctions are reflected in the revised conclusions to avoid overstating the findings.

6. Conclusions

In this study, the comparative seismic behavior of six full-scale prefabricated composite walls was investigated through low-cycle reversed loading tests, macroscopic theoretical analysis, and finite element simulations. The influence of connection type was evaluated for the MRC wall panels, whereas the influence of wall panel material was evaluated only under the U-type connection condition. Because each configuration was represented by a single specimen and SFC panels with Z-type connections were not included, the following conclusions are limited to the tested configurations.

Under the tested U-type connection condition, the wall panel material significantly affected the bearing capacity, stiffness retention, and failure characteristics. The ultimate bearing capacity of the MRC specimens was 30–35% higher than that of the corresponding SFC specimens, and their ultimate displacement was approximately 1.2–2.3 times greater. The MRC specimens retained approximately 21% of their initial stiffness and exhibited relatively controllable semi-brittle damage without pronounced overall instability. By comparison, the SFC specimens were more susceptible to local crushing and instability at displacements of approximately 35–60 mm, and their residual stiffness decreased to approximately 6–8%.

1. Within the tested configurations, the seismic response of the composite walls was governed by the sequential interaction between connection deformation and wall panel resistance. The connection type controlled when and how the wall panel participated in lateral load transfer, whereas the panel material governed the subsequent resistance, stiffness retention, and damage tolerance after engagement. The MRC walls with Z-type flexible connections exhibited promising large-deformation behavior and limited panel damage within the tested displacement range. Specimens M-Z3 and M-Z4 reached maximum tested displacements of 148.34 and 155.37 mm, respectively, while retaining approximately 85% and 100% of their recorded peak loads at test termination. In addition, the total energy dissipation of specimen M-Z3 was approximately 80% higher than that of specimen M-U1. However, the Z-type specimens did not reach the prescribed structural failure criterion because the tests were terminated by the actuator displacement limit. Therefore, their true ultimate displacement and ultimate ductility remain undetermined. Moreover, because SFC panels with Z-type connections were not tested, the universal superiority of the Z-type connection–MRC panel configuration cannot be established. Additional full-factorial and repeated tests are required before general seismic design recommendations can be proposed.
2. The bearing capacities predicted by the established macroscopic restoring-force model were generally consistent with the measured values of the tested specimens. The relative errors were within 11% for the U-type MRC specimens and within 14% for the other tested configurations. These results indicate that the proposed model provides reasonable estimates of the bearing capacity for the specimens considered in this study. However, its applicability to other wall panel materials, connection details, axial load levels, wall dimensions, and boundary conditions requires further experimental validation.

3. Within the adopted modeling assumptions, the monotonic finite element model reasonably reproduced the experimental skeleton-curve response, peak resistance, secant-stiffness evolution along the envelope response, and principal stress-concentration regions. The prediction error for the peak resistance was within 5%, and the average discrepancy in the secant-stiffness evolution was less than 11%. The numerical results therefore provide a reasonable qualitative basis for interpreting the envelope response, stress distribution, and load-transfer mechanisms of the tested configurations. However, the model was not intended to reproduce reversed cyclic behavior, including unloading and reloading paths, pinching, cycle-dependent stiffness degradation, cumulative damage, hysteretic energy dissipation, and local material crushing. The effects of friction coefficients, contact stiffness, bolt pretension, installation gaps, and other interface parameters also require further calibration and sensitivity analysis.

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