

Article

Study on the Composite Performance of Sandwich Wall Panels with SGCCs

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Abstract: This paper aims to explore the impact of different arrangements of new steel-glass FRP composite connectors (SGCCs) on the bending and composite performance of sandwich wall panels. Through monotonic loading bending tests on six full-size specimens, aspects such as their failure modes, load-deflection curves, load-strain relationships, slip between the thermal insulation layer and concrete, and composite action were analyzed. The results show that all sandwich wall panels experienced bending and ductile failure, and exhibit partial composite performance, with P4 having the best composite performance and P1 the worst. The degree of composite action is positively correlated with the flexural bearing capacity. The bending capacity mainly depends on the layout rather than the total number of SGCCs. Arranging connectors along the short side of the panel has a more significant impact, and the number of connectors at the panel's ends has a greater influence on the composite performance. Except for P1, the theoretical value of the composite degree of the other sandwich wall panels exceeds 70%, and P4 reaches 85%. The theoretical calculations are in good agreement with the experimental results. This study provides theoretical and data support for the rational configuration of connectors in sandwich wall panels and is of great significance for building engineering applications. Meanwhile, suggestions for configuring connectors in actual engineering are also given.



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Keywords: sandwich wall panels; steel-glass FRP composite connector; bending capacity; composite performance; theoretical calculation

1. Introduction

Energy conservation and emission reduction in the construction industry are key measures to achieve the goals of carbon peak and carbon neutrality. External wall insulation accounts for a large proportion of building energy-saving systems. Sandwich wall panel, also known as precast concrete sandwich insulation wall panels, consists of inner and outer concrete wythes and a middle insulation layer [1]. The inner and outer concrete wythes are usually connected using connectors (such as metal connectors, concrete blocks, or FRP connectors) through the insulation layer to form a “sandwich” overall structure [2–4]. Due to its superior thermal and structural efficiency [5–7], sandwich wall panels have received widespread attention and application in the development of building industrialization.

According to the interaction between the inner and outer concrete wythes and the insulation layer, sandwich wall panels can be divided into three types: non-composite panels, fully composite panels, and partially composite panels [1]. These categories indicate the degree to which the two concrete wythes act in unison to resist loads. Non-composite means the connector is unable to transmit force or possess stiffness, and the inner and outer

wythes bear loads independently. While fully composite behavior implies the presence of connections with infinite rigidity and enables all shear force transmission, the inner and outer wythes deform in full coordination and jointly bear the loads. Partially composite represents a state intermediate between fully composite and non-composite. In this state, the connectors can transfer a portion of the shear forces, realize the connection and stress transfer between the inner and outer wythes to a certain extent, and enable them to have a certain ability to work cooperatively. The bending strain distribution along the section height of each type of panel is shown in Figure 1. The degree of composite action mainly depends on the nature of connectors that connect the inner and outer concrete wythes.

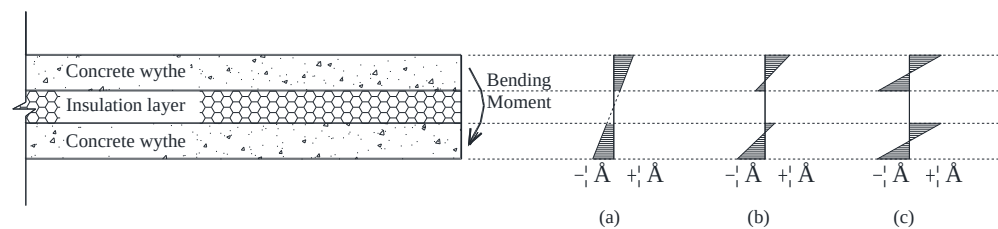


Figure 1. Strain distribution in sandwich wall panel under flexure: (a) Fully composite; (b) Partially composite; (c) Non-composite.

In high-rise buildings, sandwich wall panels provide thermal insulation shells for the building, mainly bearing vertical self-weight, horizontal wind loads, and seismic effects [5–7]. The load borne by the outer concrete wythe will be transmitted to the inner wythe through connectors. Connectors are the key components of sandwich wall panels.

Steel connectors are widely used in traditional sandwich wall panels, and their excellent mechanical properties can enable sandwich wall panels to achieve high composite action [8–10]. However, steel connectors have high thermal conductivity (45–60 W/(m·K)) and are prone to thermal bridging effects in sandwich wall panels, resulting in poor insulation performance [11], and no longer meeting current energy-saving requirements [12,13].

Fiber-reinforced polymer (FRP) has low thermal conductivity (0.2–0.3 W/(m·K)) and high tensile strength (usually 2–10 times that of steel bars). Using FRP connectors instead of traditional steel connectors can effectively solve the problem of the “thermal bridging effect” [14–17] and significantly reduce external wall energy consumption. However, the low shear strength of FRP connectors (usually 50–60 MPa and only 5–20% of the tensile strength of FRP connectors) can easily lead to interlayer shear failure of the inner and outer concrete wythes at the connection points [18,19], and their stiffness cannot ensure that the inner and outer concrete wythes of the sandwich wall panel work synergistically to achieve spatial composite action.

In practical applications of traditional sandwich wall panels, the performance of connectors has the above-mentioned limitations. Therefore, how to optimize the connectors of sandwich wall panels to meet the requirements of mechanical properties, thermal insulation performance, and durability simultaneously has become a crucial problem urgently to be solved at present, which is also the starting point of this study.

Considering the good ductility and high shear strength of the steel, as well as the excellent thermal efficiency and corrosion resistance of FRP, some scholars have attempted to combine the two to form a composite reinforcement and apply it in construction engineering.

Zheng Bailin took the initiative to propose a novel type of FRP-wrapped rebar [20], which had steel as the core with $\pm 45^\circ$ angle laminations of glass fiber impregnated with epoxy around it and which was then cured as a whole. Research showed that the mechanical deformation performance between the metal core and the FRP-wrapped layer was correlated, and the mechanical behavior of the FRP-wrapped rebar had a good bilinear character. After that, Wu Gang [21] conducted experimental and theoretical studies on the

mechanical properties of steel-FRP composite bars (SFCB). The results showed that SFCB had a high elastic modulus, high tensile strength (about 50% higher than ordinary steel bars), stable post-yield stiffness, and excellent corrosion resistance. In view of these studies, the composite material composed of a mixture of metal and non-metal can also be applied to sandwich wall panels to improve the thermal and structural performance of buildings.

The team led by Guo Zhengxing from Southeast University has independently developed a new type of connector of W-shaped Steel Glass Fiber Reinforced Polymer (SGFRP) [22]. The SGFRP connector uses steel bars as the inner core, and glass fibers impregnated with epoxy resin are wrapped around the core in a circumferential manner. Researchers conducted a series of experimental studies on SGFRP connectors and sandwich wall panels and found that such W-shaped SGFRP connectors have high tensile and shear strength [23–25]. However, due to the difficulty in controlling the anchoring depth during construction, the connectors tend to anchor improperly and be pulled out prematurely when the sandwich wall panels bend, thereby reducing the bending resistance and composite performance of the sandwich wall panels.

To solve the anchoring and positioning problem of connectors, the research team has developed another type of Steel-Glass fiber-reinforced polymer Composite Connector (SGCC), as shown in Figure 2. The steel core skeleton of SGCC is formed in the following way: first, one end of the scored steel wire or ribbed steel bar is upset, and then it is inserted into a metal disc with a hole in the middle to make a disc expansion head. A certain thickness of GFRP resin layer is wrapped around the steel core skeleton, and then through the molding process, a steel-fiber hybrid connector is formed with a disc expansion head at one end, multiple continuous inverted conical platforms on the surface, and a circular positioning ring 30 mm away from the disc expansion head. The purpose of designing the disc expansion head and inverted conical platforms is to enhance the anchoring ability of the connector, while the purpose of the positioning ring is to facilitate the positioning of the connector when inserted into the insulation layer. The SGCCs are molded using a 315T hydraulic press equipment (Ningbo Hilead Hydraulic Co., Ltd., Ningbo, China), and the preparation process is as follows: apply a release agent to the mold → filler preforming → molding → heating and solidifying → demolding → deburring. This process and the physical photo of SGCCs are shown in Figure 3. The SGCC can be set in two types: vertical insertion type and 45° oblique insertion type. As shown in Figure 4, the two types can be used in pairs to form a triangular structure to tie the three interlayers of the sandwich wall panels. The insertion direction of the oblique connector is perpendicular to the short side of the sandwich wall panels to transmit shear force.

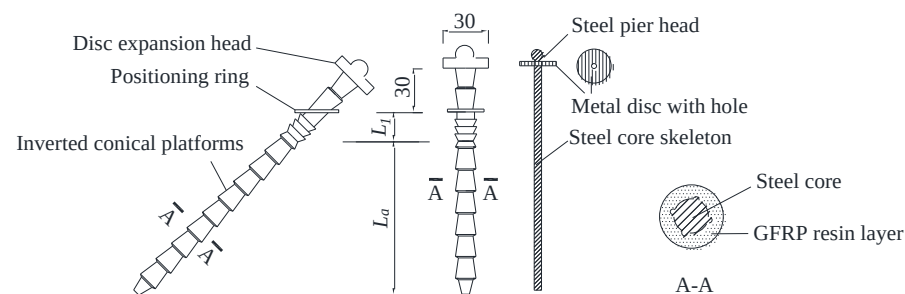


Figure 2. Constructional detail of SGCCs.

Previous studies have been conducted on the mechanical properties of SGCC and the flexural and composite performance of sandwich wall panels when uniformly arranged [26]. However, the influence of different arrangements of SGCCs on the bending strength and composite performance of sandwich wall panels is still unclear.

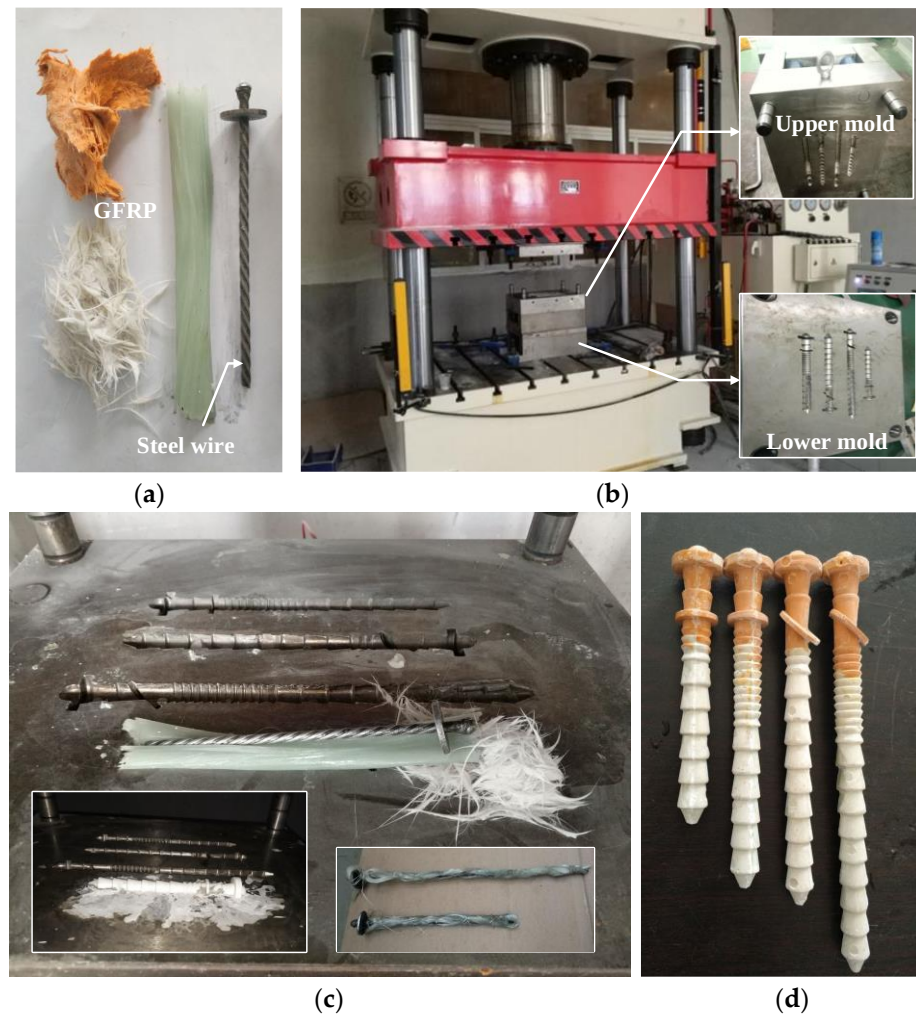


Figure 3. The preparation process of SGCCs: (a) Raw materials; (b) 315T hydraulic press equipment and molds; (c) Filling, molding, forming and demolding; (d) Physical photo of SGCCs.

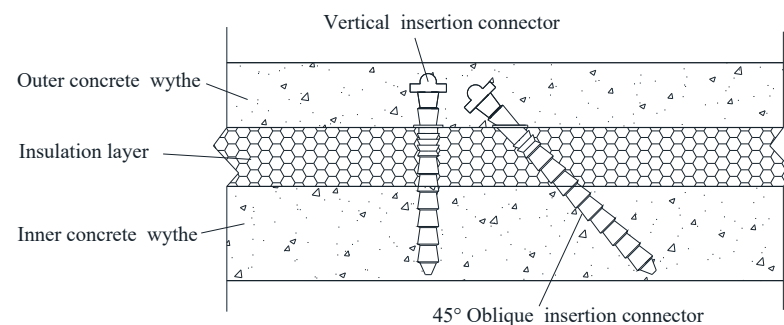


Figure 4. Components of sandwich wall panel with SGCCs.

Currently, there is limited research on the effects of different arrangements of connectors on the bending strength and composite performance of sandwich wall panels. Existing studies mainly focus on the performance of panels with uniformly spaced connectors [27–34], lacking a comprehensive and systematic comparative analysis of various connector arrangements. This study conducted monotonic loading bending tests on sandwich wall panels with different arrangements of SGCCs and evaluated their composite performance, filling the gap in research on the relationship between connector arrangement and panel performance in this field. It provides theoretical basis and data support for the reasonable arrangement of connectors in sandwich wall panels and helps to promote the further optimization and application of sandwich wall panels in construction engineering.

2. Experimental Program

2.1. Test Specimen Design

There are a total of six-panel specimens for the flexural test, including five sandwich wall panels and one concrete solid wall panel. The dimensions of each panel are 3200 mm $\times$ 1200 mm $\times$ 160 mm. The sandwich wall panels consist of a 50 mm thick outer concrete wythe, a 50 mm thick extruded polystyrene (XPS) insulation layer, and a 60 mm thick inner concrete wythe. The SGCC used in the sandwich wall panel specimen has a diameter of 10 mm and is composed of high-strength scored steel wire with a diameter of 5 mm and a GFRP wrapping layer with a thickness of 2.5 mm. The concrete used in the experiment is fine aggregate concrete with a strength of C40, and the compressive strength of the XPS insulation layer is 266 Kpa. All sandwich wall panels are equipped with HRB400 grade single-layer steel mesh with a diameter of 8 mm and a spacing of 200 mm in the inner and outer concrete wythes, and double-layer steel mesh is installed in the solid concrete wall panel. The dimensions and reinforcement details of each specimen are shown in Figure 5. According to the material performance test, the relevant strength indicators of the steel bars and connectors are measured in Table 1.

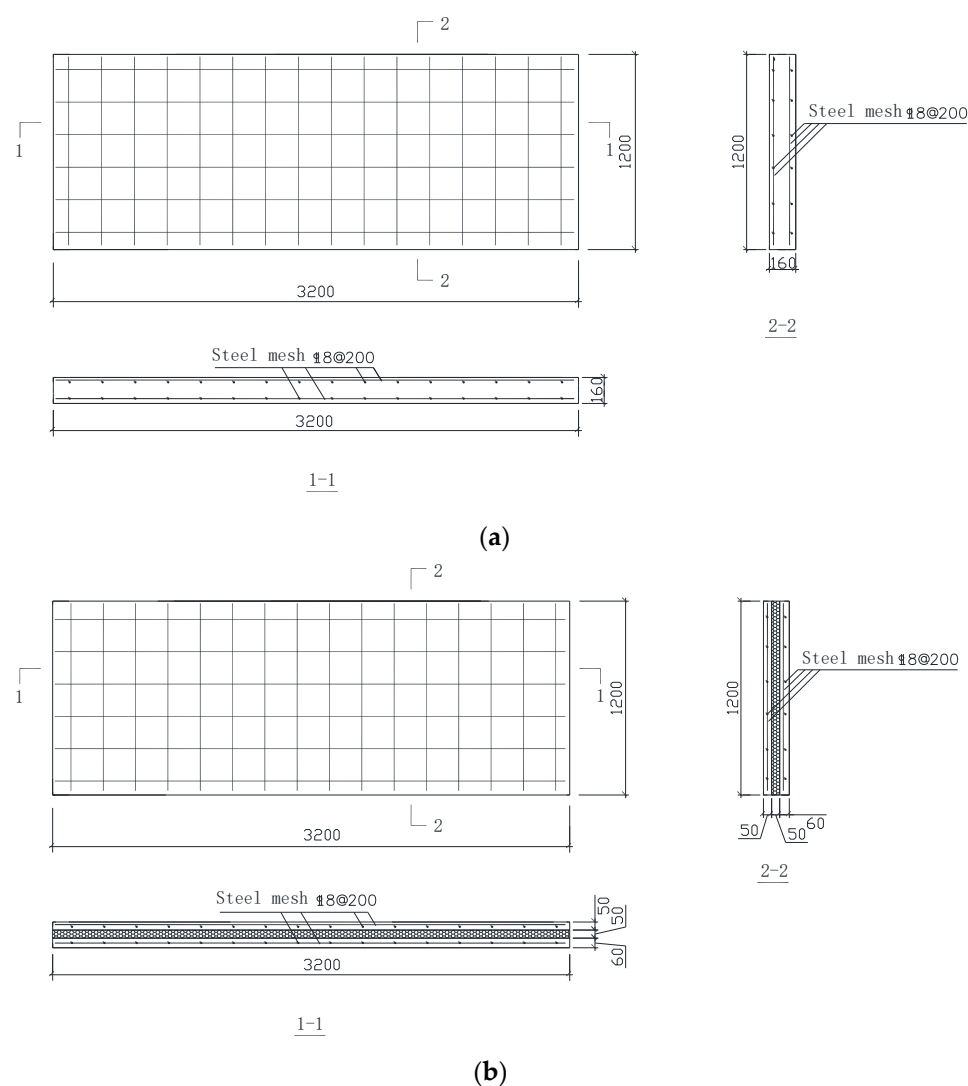


Figure 5. Reinforcement diagram of flexural test panels (all dimensions in mm): (a) Solid concrete wall panel; (b) Sandwich wall panel.

Table 1. Mechanical properties of materials.

Material Type	Diameter (mm)	Elastic Modulus (GPa)	Yield Strength (MPa)	Ultimate Strength (MPa)	Shearing Strength (MPa)
steel	8	200	495	656	--
SGCC	10	59	357	442	409

The five full-size sandwich wall panels use five different arrangements of SGCCs, as shown in Figure 6. The first panel P1(2-1-1-2) has two pairs of connectors at both ends, with one pair arranged in the middle. In the second panel P2(2-2-2-2), two rows of connectors are arranged along the short side, with two pairs at each end and middle section of the panel. Three pairs of connectors are arranged at both ends of the third panel P3(3-2-2-3), and two pairs of connectors are arranged in the middle section. The fourth panel P4(3-3-3-3) is based on the layout of P2(2-2-2-2), with the spacing between connectors reduced along the short side of the panel, and three pairs of connectors are arranged at both ends and the middle part of the panel. The fifth panel P5(2-2-2-2-2-2) is based on the layout of P2(2-2-2-2), with the spacing between connectors tightened along the long side of the panel. Two pairs of connectors are arranged at each end, quarter, and middle of the panel, that is, a total of six pairs of connectors are arranged in each row parallel to the long side direction. The purpose of arranging different numbers of connectors at different positions on the sandwich wall panel is to understand the influence of different connector layouts and quantities on the bending capacity and composite performance of sandwich wall panels, in order to find an optimal connector arrangement scheme. For the convenience of expression and to enhance the readability of the paper, each test panel will be abbreviated as P0, P1, P2, P3, P4, and P5 respectively in the following text.

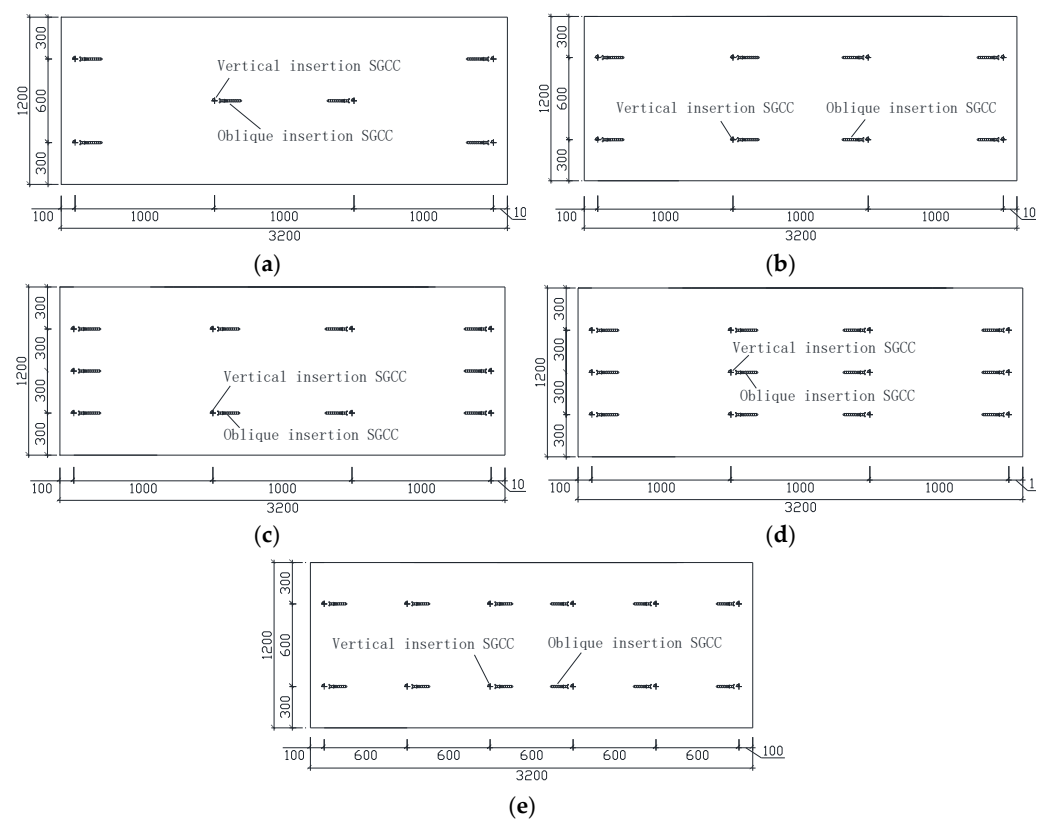


Figure 6. Plan view of the five test sandwich wall panels (all dimensions in mm): (a) P1 (2-1-1-2); (b) P2 (2-2-2-2); (c) P3 (3-2-2-3); (d) P4 (3-3-3-3); (e) P5 (2-2-2-2-2-2).

2.2. Preparation of Test Specimens

The production of sandwich wall panel specimens starts from the XPS insulation layer. Cut the XPS insulation layer and splice it together to match the width of the panel. Mark the position of the connectors on the surface of the XPS insulation layer according to the design, and then place them aside for future use.

The sandwich wall panel specimens are made using the forward striking process of inner concrete wythe, insulation layer and outer concrete wythe. After the steel formwork is prepared, the steel mesh is in place, concrete is poured and compacted by vibration to form the inner concrete wythe. Lay the prepared XPS insulation layer on the upper surface of the inner concrete wythe in order, insert SGCCs according to the marked points, perform self-positioning through the positioning ring on the connectors, and insert the lower end of the connectors into the inner concrete wythe. Then fill the gap of the insulation layer with canned expansion foam, and scrape off the excess foam. After ensuring that all connectors are inserted and positioned, place the upper steel mesh in place, pour the upper outer concrete wythe, and compact it by vibration. Finally, smooth and finish the surface. The forward striking process can firmly embed the enlarged head of SGCCs into the outer concrete wythe, providing a stronger anchoring effect. In addition, three cubic test blocks were prepared while the concrete was being poured. The production process of sandwich wall panels with SGCCs is shown in Figure 7.

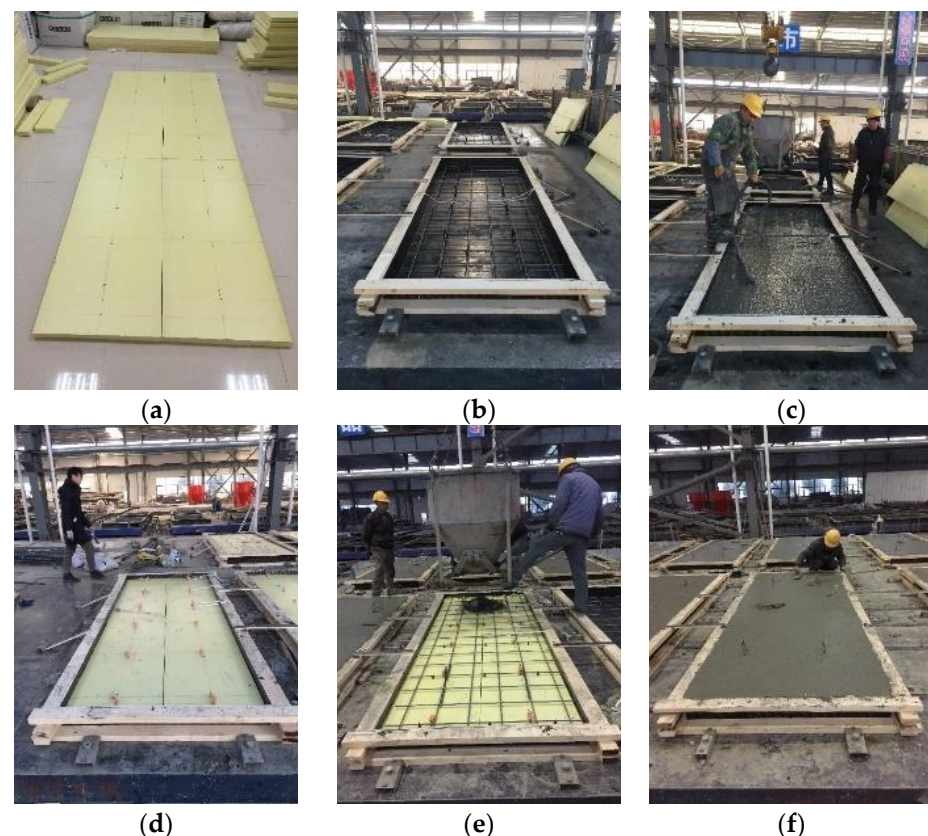


Figure 7. Fabrication and casting process of sandwich wall panels: (a) Cutting and splicing XPS insulation layer; (b) Laying steel mesh of inner concrete wythe; (c) Pouring and vibrating inner concrete wythe; (d) Arranging insulation layer and inserting SGCCs; (e) Laying the steel mesh of outer wythe and pouring concrete; (f) Smoothing and finishing the concrete surface.

2.3. Test Setup

The experiment adopts a four-point support and two-point symmetrical loading method. When placing the sandwich wall panel, the inner wythe is placed below and the

outer wythe is placed above. The test setup is shown in Figures 8 and 9. The net span of all wall panels is 2900 mm, and a load distribution beam is used to distribute the load onto two I-shaped line load beams, forming a 600 mm pure bending section in the middle of the wall panel and an 1150 mm bending shear section on both sides. The panel specimens were tested and calibrated simultaneously using a manual digital pull-out instrument with a range of 30T and a force sensor with a range of 20T.

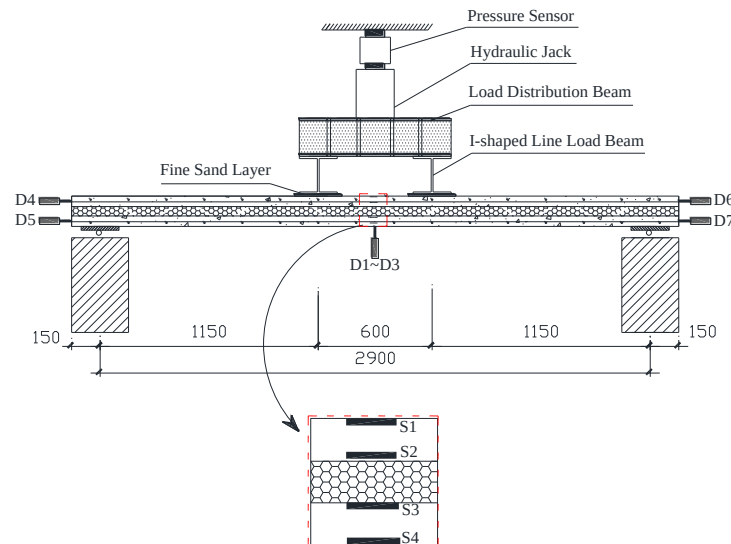


Figure 8. Design drawing of test setup (unit: mm).



Figure 9. Test setup at the scene.

2.4. Test Content and Process

A total of seven displacement gauges and four strain gauges were installed on each panel. Three displacement gauges (D1–D3) with a range of 300 mm were installed at the bottom of the mid-span of the panel to measure the deflection at the mid-span. Four displacement gauges with a range of 100 mm were installed in the middle of the inner and outer concrete wythes at both ends of the panel (D4 and D5 on the west side, D6 and D7 on the east side) to measure the relative slip between the inner and outer concrete layers, as shown in Figure 8. Four concrete strain gauges (S1–S4) with a length of 50 mm were pasted on the side of the mid-span of the panel to measure the strain along the section height of the panel. The positions of the strain gauges are shown in Figure 8. In addition, in order to understand the efficiency of SGCCs at different positions of the panel in the bending

process, strain gauges with a length of 5 mm were pasted on the connectors at the end, quarter, and middle positions of panel P5.

Collect initial readings before placing the line load beams to deduct the deflection of the panel under its weight. Before formal loading, pre-load to 5 kN to ensure that all instruments and equipment are in close contact and working properly, and then unload to zero. The experimental loading is controlled by force, with each level being loaded in steps of 2 kN and held for 5 min until the load drops to 85% of the peak load and cannot rise again. This marks the complete failure of the panel and the loading is now terminated. During loading, the DH3816Net (V3.4.04) static strain testing system (DongHua Testing Technology Co., Ltd., Taizhou, China) is used to continuously collect and record data at 1-s intervals. At each level of the load stage, the crack morphology of the panel should also be recorded, the trend of the cracks should be traced with a marker pen, and the corresponding load value should be indicated. At the same time, the width of cracks and the slip between the insulation layer and the concrete wythes at both ends of the panels under each level of load is measured with a steel ruler and recorded.

2.5. Experimental Phenomena and Destructive Characteristics

All test panels experienced bending failure, and the failure process of each sandwich wall panel was very similar. The first full-length crack appeared in the pure bending section at the bottom of the inner concrete wythe and extended from the bottom to the side; As the load increased, new cracks continued to appear at the bottom of the inner concrete wythe, and the cracks on the panel side gradually extended upwards. Cracks also began to appear on the outer wythe side near the position of the line load beam, and slight slippage occurred between the insulation layer and the concrete layers. As the load further increased, the crack width also increased, and new cracks continued to appear on the outer concrete wythe side. The insulation layer broke, and there was obvious slippage between the insulation layer and the concrete layers. The deflection of the panel at the mid-span gradually increased. Finally, the surface of the outer concrete wythe cracked or the concrete was crushed, the crack width rapidly increased, and the deflection of the panel at the mid-span increased sharply. After the load dropped to 85% of the peak load and it could not continue to be loaded, the specimen was declared damaged and the test was stopped. After the experiment, the insulation material at the panel end was removed, and it was found that some SGCCs had fiber splitting and concrete anchorage failure. The entire failure process of all sandwich wall panels was slow, with significant deformation, all of which belonged to ductile failure. The crack distribution and failure mode of the test panels are shown in Figures 10 and 11, respectively.

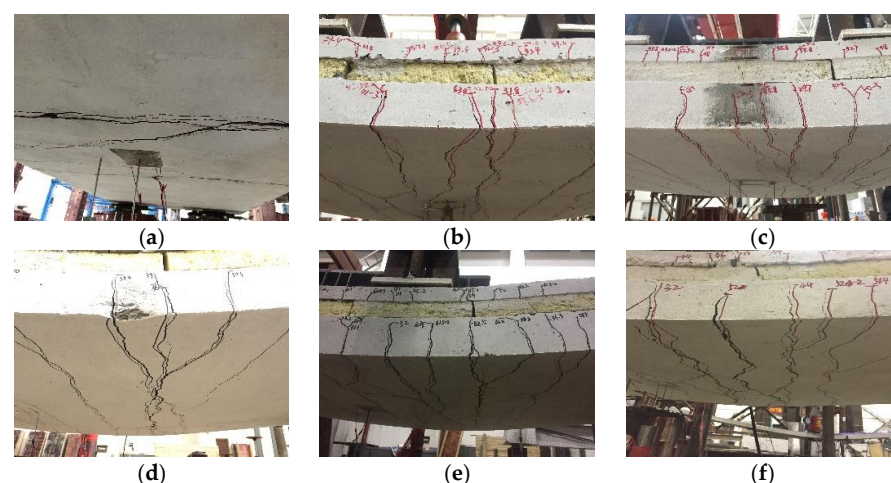


Figure 10. Crack distribution at the bottom of all test panels: (a) P0; (b) P1; (c) P2; (d) P3; (e) P4; (f) P5.

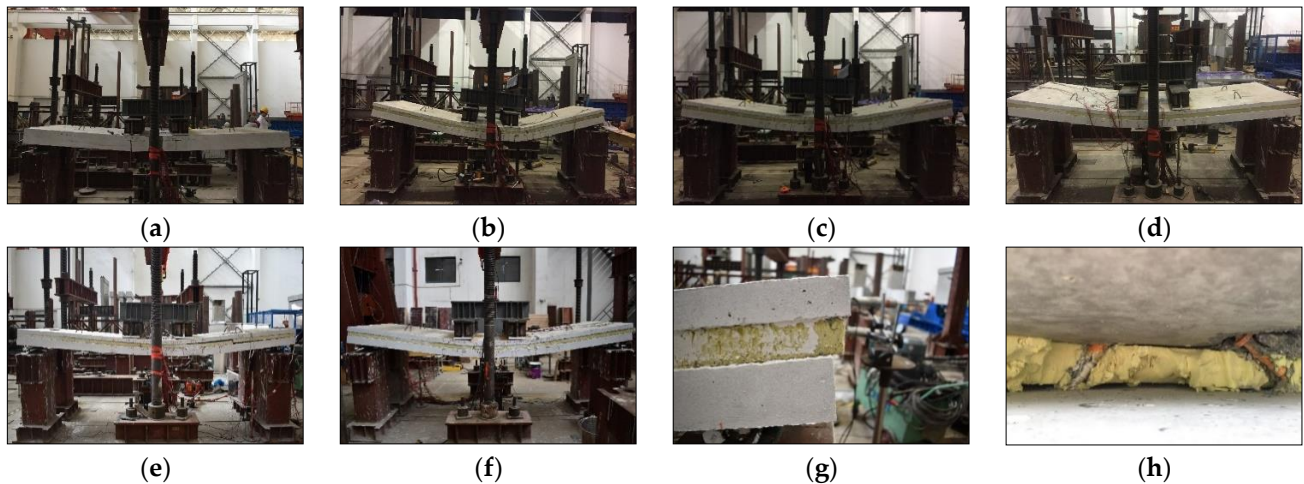


Figure 11. Failure modes of all test panels: (a) P0; (b) P1; (c) P2; (d) P3; (e) P4; (f) P5; (g) Relative interlayer slip at the panel end; (h) Failure mode of SGCCs at the panel end.

3. Results and Discussion

Based on the load-deflection curve, the load-strain relationship of connectors, strain variation along the section height, and load-relative slip relationship, the obtained experimental results were analyzed and discussed.

3.1. Load-Deflection Curve

The load-deflection curves of each flexural specimen are shown in Figure 12.

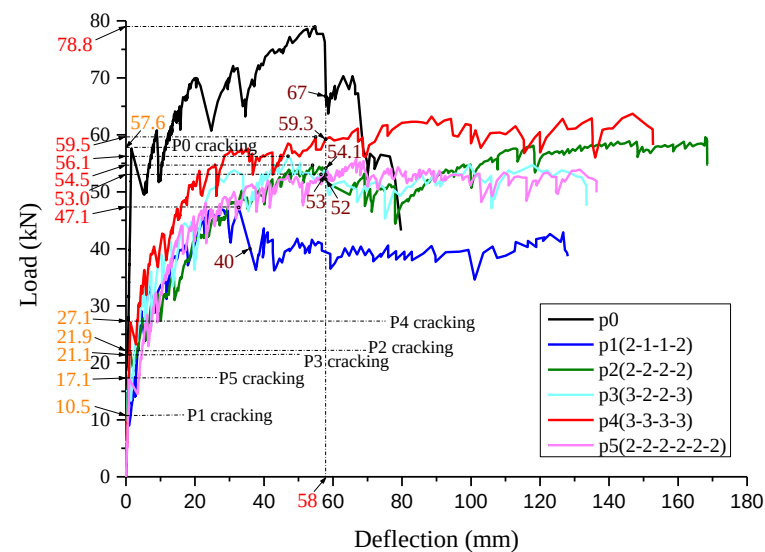


Figure 12. Load-deflection curves for test panels at mid-span.

From the load-deflection curve, it can be seen that during the loading process, the load of each specimen has varying degrees of fluctuations in both decrease and increase, and it has undergone a long process from loading to complete failure. The load-deflection curve of panel P1 tends to flatten after decreasing to 85% of the peak load, at which point the panel can still withstand a certain load, but with significant deformation. The load-deflection curves of panels P2 and P3 will slowly rise back after decreasing to 85% of the peak load, until reaching a higher load point, and then sharply decline, causing significant deformation of the panels. The load-deflection curve of panel P4 rises gently after the yielding of the panel, until the maximum load is reached and the load begins to decrease, resulting in significant deformation of the panel. The load-deflection curve of panel P5 gradually

risers to the peak load and then decreases, and tends to flatten before reaching 85% of the peak load. At this point, the panel continues to bear the load, but undergoes significant deformation. In previous studies [31,32], the load reduction trend of test sandwich wall panels when reaching the ultimate state was relatively single. However, in this study, the load changes of each panel showed diverse fluctuation characteristics, which was different from previous studies. This is mainly attributed to the influence of different arrangements of connectors, which further highlights the complexity and uniqueness of the impact of reasonable arrangement of SGCCs on the mechanical properties of panels.

According to Article 9.3.2 of the Chinese standard “Code for Acceptance of Construction Quality of Concrete Structures” (GB50204-2015) [35], when the maximum crack width at the tensile main reinforcement of a bending member reaches 1.5 mm or the deflection reaches 1/50 of the span, the member is considered to have reached the ultimate bearing capacity state. In this experiment, considering the deformation of the panel, crack development, and the characteristics of the load-deflection curve, it is believed that even if the bearing capacity of the panel has not decreased to 85% of its peak value, as long as the mid-span deflection of the panel exceeds 58 mm ($l_0/50$), the panel has also reached its ultimate bearing capacity state.

The characteristic values on each load-deflection curve during the loading process are listed in Table 2, and all characteristic values are taken from the curve corresponding to the deflection at the mid-span of the panel before reaching 58 mm. In Table 2, P_{cr} , P_y , P_{max} , and P_u represent the loads corresponding to the cracking point, yield point, peak point, and ultimate point, respectively, while Δ_{cr} , Δ_y , Δ_{max} , and Δ_u represent the corresponding displacements. The yield point is obtained through a geometric drawing method [36]. The ultimate point refers to the point where the bearing capacity of the specimen decreases to 85% of the peak load during the loading process. If it does not decrease to 85% of the peak load, the load value corresponding to the ultimate deflection of 58 mm is taken.

Table 2. Characteristic values of the load-deflection curves.

Specimens	Cracking Point		Yield Point		Peak Point		Ultimate Point	
	P_{cr} (kN)	Δ_{cr} (mm)	P_y (kN)	Δ_y (mm)	P_p (kN)	Δ_p (mm)	P_u (kN)	Δ_u (mm)
P0	57.6	1.57	69.7	19.59	78.8	54.86	67	57.89
P1	10.5	0.58	39.7	15.36	47.1	32.54	40.0	35.98
P2	21.9	1.24	50.5	37.97	54.5	54.11	52.0	58
P3	21.1	1.01	50.7	25.71	56.1	47.03	54.1	58
P4	27.1	1.27	57.4	30.48	59.5	55.02	59.3	58
P5	17.1	0.96	45.7	19.66	53.0	57.62	53	58

The cracking load, peak load, and ultimate load values of each test panel are also clearly marked in Figure 12 for easy reference and comparison.

From the load-deflection curve, it can be observed that before cracking, each panel exhibits an approximately linear increase in deflection with increasing load. However, after the panel cracks, the load-deflection curve demonstrates nonlinear behavior, indicating a more complex relationship between load and deflection. The long time from yielding to failure of the panel indicates that its ductility is excellent. According to the data in Table 2, the initial stiffness (ratio of cracking load to cracking deflection) of the six test panels was calculated to be 36.7, 18.1, 17.7, 20.9, 21.3, and 17.8, respectively. The initial stiffness of all sandwich wall panels is generally higher than that in the existing studies [32,37]. This might be because the SGCCs used in this study are more conducive to improving the overall

collaborative performance of the panels in structural design, thus enabling the panels to resist deformation more effectively when subjected to initial loads. In addition, the fact that the initial stiffness of panels P1, P2, and P5 is close, while the initial stiffness of panels P3 and P4 is also close and relatively high indicates that the initial stiffness of these panels is positively correlated with the number of connectors at the end of the panels.

In the load-deflection curve graph, solid concrete wall panel P0 is considered as a fully composite panel to provide a reference for the composite performance of sandwich wall panels. It can be seen that, compared to other sandwich wall panels, the load-deflection curve of panel P4 is significantly higher and closest to the P0 curve. It exhibits higher bending capacity and composite performance. The load-deflection curve of P3 is slightly lower than that of P4, and the composite performance of P3 is inferior to that of P4. The load-deflection curves of P2 and P5 are very similar, while the curve of panel P1 is located at the very bottom, farthest from the P0 curve, exhibiting the lowest composite performance.

Comparing the load-deflection curves of P2, P4, and P5, it can be inferred that adding the same number of connectors along the short side of the panel has a more pronounced effect on its bending capacity and composite performance than adding them along the long side. In other words, reducing the spacing between connectors along the short side of the panel has a more significant effect on its bending capacity and composite performance than reducing the spacing between connectors along the long side. Meanwhile, it should be noted that panels P4 and P5 have the same number of connectors, but P4 exhibits a 10.6% higher bending capacity than P5, indicating that their bending and composite performance characteristics are not identical. On the other hand, although P5 has 33% more connectors than P2, their bending capacities differ by less than 2%. This indicates that the bending resistance and composite performance of sandwich wall panels are not proportional to the total number of connectors, but are mainly influenced by the layout of connectors.

In actual engineering projects, for the sandwich wall panels of high-rise buildings or those bearing relatively large loads on the exterior walls of buildings, it is recommended to preferentially adopt a connector arrangement method similar to that of P4. That is, appropriately densify the connector spacing in the short-side direction of the panel to enhance the flexural and composite performance of the panel. Meanwhile, attention should be paid to the quantity and arrangement of the connectors at the end of the panel. Since the connectors at the end have a significant impact on the composite performance, a sufficient quantity and reasonable anchorage depth should be ensured. For example, the composite performance of the P3 and P4 panels, which are equipped with more connectors at the end, is superior to that of other panels. In addition, when determining the quantity and layout of the connectors, it is necessary to combine the structural design calculation results of specific projects and comprehensively consider factors such as the height, number of floors, wind load, and seismic fortification intensity of the building, and conduct optimized design to avoid blindly increasing the quantity of connectors while neglecting the rationality of the layout, so as to ensure the safety and stability of the sandwich wall panels during long-term use.

3.2. Strain of SGCCs

The attachment position of strain gauges on SGCCs in P5 and the load-strain relationship are shown in Figure 13. In the figure, Z1, X1, Z2, X2, Z3, and X3 respectively represent strain gauges on the vertical insertion and oblique insertion connectors at the end, quarter, and middle positions of the panel.

It can be observed that the strain in the connectors cannot be ignored before the panel reaches its ultimate load. For the connectors at the end and quarter of the panel, they mainly bear tensile strain, and the strain value changes approximately linearly with the increase in load. For the connectors in the middle of the panel, they mainly bear compressive strain

in the early stage and gradually transform into tensile strain in the later stage. At the same time, the tensile stress of connectors at the end is significantly greater than that at the quarter position, and the tensile stress of the oblique insertion connectors is greater than that of the vertical insertion connectors. The above phenomena indicate that under bending, the interlayer shear force between the inner and outer concrete wythes of the sandwich wall panel is mainly transmitted through the oblique insertion connectors. The connectors at the panel end exert the greatest resistance to bending, indicating that the bending stiffness of the panel is mainly determined by the stiffness provided by the connectors at the ends of the panel. This further confirms the conclusion in Section 3.1.

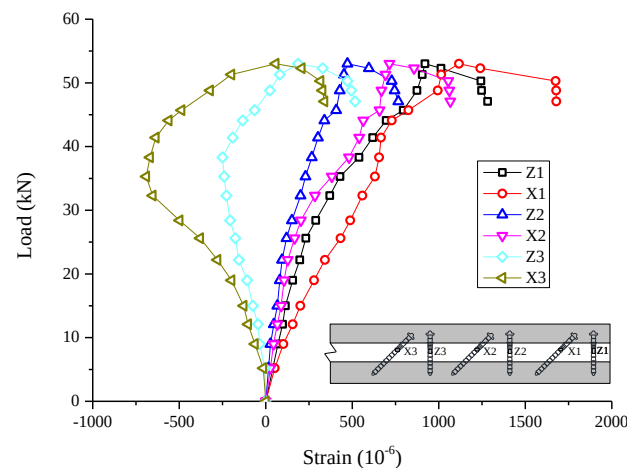


Figure 13. Strain of SGCCs at different positions in panel P5.

3.3. Strain Variation Along the Section Height

Figure 14 shows the strain variation along the section height of six test wall panels at mid-span at different load stages.

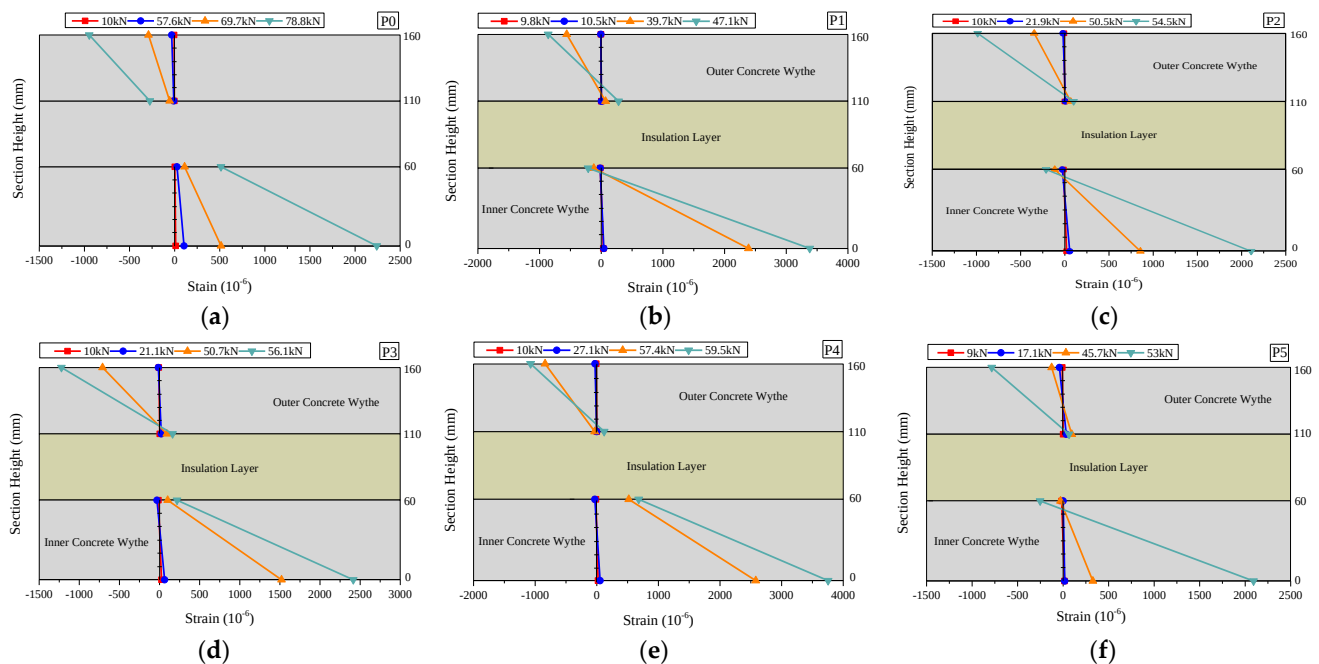


Figure 14. Strain variation along section height of the test panels at mid-span at different load stages: (a) P0; (b) P1; (c) P2; (d) P3; (e) P4; (f) P5.

In the initial loading stage, the strain of two concrete wythes was small, and the strain variation of all panels along the section height was close to continuous and basically linearly distributed, which conforms to the assumption of a flat section. The composite performance of the inner and outer concrete wythes was good. After the panel cracked, the discontinuity of strain between the inner and outer concrete wythes began to emerge and gradually became significant as the load increased because the composite action would degrade with the cracking of wythes, the sliding between the insulation layer and concrete wythes, the yielding of the steel mesh, and the damage of the connectors.

It can be observed that sandwich wall panels exhibit partial composite behavior. The strain variation level of panel P4 is closest to that of panel P0, indicating that the composite performance of panel P4 is the best. Compared with other panels, panel P1 features a more conspicuous strain discontinuity between the layers. This panel exhibits weaker composite performance and tends to exhibit non-composite behavior, as both the inner and outer concrete wythes approximately form their neutral axes. Compared with panel P2, the strain variation of panel P3 is closer to that of P4, indicating that adding the same number of connectors at the end of the panel is more significant in improving the composite performance of sandwich wall panels than in the middle of the panel. The strain variation of panel P5 is similar to that of P2, and the discontinuity is more pronounced under the ultimate loading state. In summary, the degree of composite action of sandwich wall panels is not directly proportional to the total number of connectors but mainly depends on the distribution and spacing of connectors. This also confirms the conclusion described in the previous section.

3.4. Load-Relative Slip Relationship

In the initial bending load stage, the interlayer shear force of sandwich wall panels was mainly resisted by the connectors in the bending shear section and the interfacial bonding force between the XPS insulation layer and concrete layers. After bending cracks occurred, as the load increased, the XPS insulation layer gradually separated from the concrete layer until the insulation layer fractured, as shown in Figure 15.

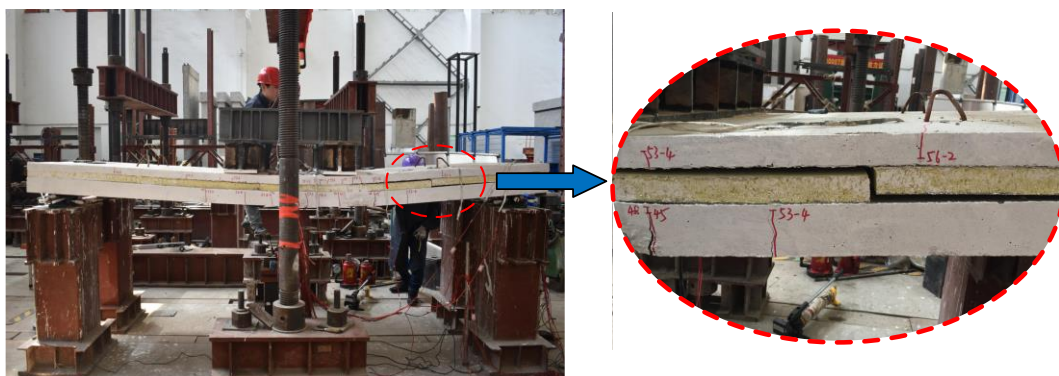


Figure 15. Interface debonding between XPS insulation and concrete wythes and XPS insulation layer fracture.

The initial bonding failure mainly occurred at the interface between the insulation layer and the bottom concrete layer near the quarter-span position of the panel, and then sliding began at the interface. As the bending deflection increased, the insulation layer experienced brittle fracture, followed by debonding between the insulation layer and the upper concrete layer. At this stage, most of the shear force was transmitted by the connectors, and the sandwich wall panels exhibited a certain degree of composite action.

As the load continued to increase, significant relative slippage began to occur between the upper and lower concrete layers and the insulation layer.

Figure 16 shows the relationship curve between the load and the total relative slip at the end of the inner and outer concrete wythes before the sandwich wall panel reaches the ultimate bearing capacity state.

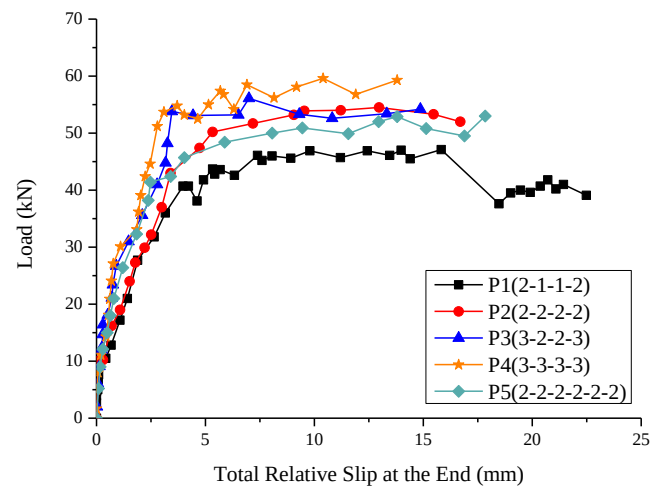


Figure 16. Load-total relative slip relationships of test sandwich wall panels.

The total relative slip at the end of each panel represents the sum of the relative displacement between the inner and outer concrete wythes and the insulation layer at the left and right ends of the panel. The total relative slip at the end of the panel can, to some extent, reflect the degree of partial composite action. When the total relative slip at the end is small, it indicates that the collaborative performance between the layers of the sandwich wall panel is good and can effectively share the load, which means that the degree of partial composite action is high. On the contrary, when the total relative slip at the end is large, it indicates poor collaborative performance between the layers and a lower degree of partial composite action.

From Figure 16, it can be seen that under ultimate load, the total relative slip at the end of each test panel is ranked from small to large as $P4 < P3 < P2 < P5 < P1$, indicating that the degree of composite action of each panel decreases in order. Furthermore, it is indicated that the composite performance of sandwich wall panels mainly depends on the number and layout of connectors at the ends of the panels.

In practical engineering applications, to ensure the stability and safety of sandwich wall panels during long-term use, the quantity and layout of connectors at the ends of the panels should be reasonably determined according to specific engineering requirements and structural designs. For regions with relatively large wind loads or high seismic fortification intensities, the number of connectors at the ends should be appropriately increased, and the anchorage depth should be ensured to meet the design requirements, so as to enhance the overall shear resistance and composite performance of the panels. At the same time, during the construction process, the installation quality of the connectors should be strictly controlled to ensure their accurate positions and reliable anchorage and to avoid connector failures caused by improper construction, which would affect the mechanical properties and service functions of the sandwich wall panels.

4. Theoretical Analysis

The degree of composite action between the inner and outer concrete wythes directly affects the ultimate flexural bearing capacity of the sandwich wall panel. Conversely, the ultimate load can also be used to evaluate the degree of composite action of sandwich

wall panels. Referring to reference [37], we can evaluate the degree of composite action of sandwich wall panels from the perspective of strength using Equation (1). Assuming that the ultimate bending capacity corresponding to the ultimate load of the solid concrete wall panel P0 represents the ultimate bending capacity of the fully composite panel, while the ultimate bending capacity of non-composite sandwich wall panels is calculated using Equations (2)–(4) according to the theoretical method in reference [23].

$$\kappa = \frac{P_e - P_{\text{non}}}{P_{\text{full}} - P_{\text{non}}} \times 100(\%) \quad (1)$$

$$P_{\text{non}} = \frac{8M_{\text{Nu}}}{l} \quad (2)$$

$$M_{\text{Nu}} = F_s \left(h_{01} - \frac{x}{2} \right) + F_s \left(h_{02} - \frac{x}{2} \right) \quad (3)$$

$$F_s = F_c, F_s = A_s f_y, F_c = f_c b x, x = 0.8x_c \quad (4)$$

κ is employed to signify the degree of composite action achieved by the sandwich wall panel. This parameter precisely reflects the extent to which the inner and outer concrete wythes cooperate in a composite fashion within the panel, offering a quantitative measure of their combined effectiveness.

P_{full} serves to represent the experimental ultimate load value under fully composite action. This value is obtained through experimental procedures and showcases the maximum load capacity that the panel can bear when it functions as a fully composite structure.

P_{non} is designated to denote the theoretically calculated ultimate load value under non-composite action. Such a calculation is rooted in established theoretical frameworks and relevant computational principles, aiming to estimate the peak load the panel could endure in non-composite behavior.

P_e represents the experimental ultimate load value of the test specimen. It captures the actual maximum load that the specific specimen is capable of withstanding during the testing process, providing real-world data for analysis.

M_{Nu} is used to represent the ultimate bending moment of the sandwich wall panel under non-composite action.

F_s stands for the force in tensile steel bars in the inner and outer concrete wythes of the non-composite panels.

F_c denotes the compressive force on the cross-section of the inner and outer concrete wythes of the non-composite panels.

f_y represents the yield strength of the tensile steel bars.

f_c signifies the design value of the axial compressive strength of the concrete.

l refers to the net span of the panel.

A_s represents the cross-sectional area of the tensile steel bars.

b is the cross-sectional width of the panel.

h_{01} and h_{02} are utilized to represent the effective heights of the cross-section of the inner and outer concrete wythes of the sandwich wall panel.

x_c represents the height of the neutral axis of the inner and outer concrete wythes of the sandwich wall panel.

x stands for the height of the concrete compression zone of the equivalent rectangular.

Upon conducting calculations based on the above equations and parameters, the estimated degree of composite action of the test sandwich wall panels can be found in Table 3.

Table 3. Degree of composite action of sandwich wall panels.

Specimens	P_e (kN)	P_{full} (kN)	P_{non} (kN)	κ , %
P1	40.0	67	15.7	47.4
P2	52.0			70.8
P3	54.1			74.7
P4	59.3			85.0
P5	53			72.7

It can be clearly seen from the calculation results that, except for panel P1, the composite action degrees of other sandwich wall panels are all higher than 70%, which indicates that most of the panels exhibit good composite performance under the test conditions. Among them, the composite action degree of panel P4 reaches 85%, which is at a relatively high level. This shows that the arrangement and structural design of its connectors enable the synergistic work effect between the inner and outer concrete wythes to be remarkable so that they can resist bending loads more effectively and thus achieve a relatively high degree of composite action. The composite action degrees of panels P2 and P5 are similar, which indicates that although the total number of connectors of P5 is 33% more than that of P2, their similar connector layouts result in similar flexural capacities and composite performances. In addition, the composite action degree of P5 is 12.3% lower than that of P4 containing the same number of connectors, which is consistent with the fact that the flexural capacity of P5 is 10.6% lower than that of P4 in the experiment. It means that the flexural capacity and composite performance of the panels are not proportional to the total number of connectors, but are mainly influenced by the layout of the connectors. However, the composite action degree of panel P1 is the lowest, only 47.4%, which indicates that such a connector layout fails to fully exert the synergistic effect between the inner and outer wythes, thus affecting the overall composite performance.

This result is highly consistent with the conclusion drawn from the test phenomena and data in Section 3.4, further verifying our understanding of the composite performance of sandwich wall panels. Namely, the sandwich wall panels all exhibit partial composite performance during the actual loading process, and there is an obvious positive correlation between the composite action degree and the flexural bearing capacity. This implies that optimizing the layout scheme of connectors and improving the composite action degree of the panels, is expected to significantly enhance their flexural bearing capacity, thus providing more reliable structural performance guarantees for the application of sandwich wall panels in construction projects. Meanwhile, the good agreement between the theoretical calculation results and the test results also proves the accuracy and reliability of the theoretical analysis methods and calculation formulas adopted in this paper, providing powerful theoretical support for further in-depth research and optimization of the structural design of sandwich wall panels in the future.

5. Conclusions

This article discussed the composite performance and bending resistance of five sandwich wall panels with different numbers and distributions of SGCCs based on the analysis of the bending test results. Furthermore, the degree of composite action of sandwich wall panels was estimated. The conclusion is as follows:

1. During the test process, all test panels underwent bending failure and generally exhibited ductile failure characteristics. The failure process was relatively slow and accompanied by large deformations. Differences existed in the flexural bearing capacity

and composite performance among sandwich wall panels with different arrangements of connectors. Among all the sandwich wall panels, P4 showed the highest flexural bearing capacity and composite performance, followed in sequence by P3, P2, and P5, while the performance of panel P1 was the lowest.

2. By comparing the influence of different connector arrangement methods on the performance of sandwich wall panels, it was found that when the same number of connectors was added, increasing them along the short side of the wall panel had a greater impact on the flexural bearing capacity and composite degree of the panel than along the long side. That is, reducing the spacing of connectors along the short side of the panel had a more significant effect on improving the flexural performance and composite performance of the panel. At the same time, it indicated that the flexural capacity and composite performance of the sandwich wall panel were not proportional to the total number of connectors but mainly depended on the layout of the connectors.
3. Under the bending action of the sandwich wall panel, the SGCCs played a crucial role. The connectors at the end and at the $1/4$ position mainly bore tensile stress, and the tensile stress of the end connectors was greater than that at the $1/4$ position. The tensile stress of the oblique insertion connectors was greater than that of the vertical insertion connectors. Consequently, it meant that the shear force between the inner and outer concrete wythes was mainly transmitted through the tensile force borne by the oblique insertion connectors, and the flexural stiffness of the panel was mainly determined by the stiffness provided by the connectors at the end of the panel.
4. When the same number of connectors was added, increasing them at the end of the panel had a more significant impact on the composite performance than adding them in the middle of the panel.
5. All sandwich wall panels exhibited partial composite performance. Except for panel P1, the composite action degree of the other sandwich wall panels was higher than 70%. Among them, the composite action degree of panel P4 reached 85%, and that of panels P2 and P5 was similar. The theoretical evaluation result was consistent with the conclusion drawn from the test phenomena and related relationships. The composite action degree was positively correlated with the flexural bearing capacity.

In actual construction, the connectors should be reasonably configured according to factors such as the type of building, the wind load and the seismic fortification intensity of the region where the building is located. This includes preferentially adopting an arrangement method similar to that of P4, densifying the connectors along the short side of the panel, paying attention to the quantity and layout of the connectors at the ends of the panel, and ensuring the installation quality of the connectors during the construction process. The aim is to better apply the experimental research results to the actual engineering construction and ensure the mechanical properties and service functions of the sandwich wall panels.

In summary, this research has clearly elucidated the influence of different arrangements of SGCCs on the flexural strength and composite performance of sandwich wall panels through tests and theoretical analysis. It can provide a scientific basis for the rational arrangement of connectors in sandwich wall panels in practical engineering and help to further optimize the application of such panels in building envelopes.

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