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Shear-resistant behavior of light composite shear wall

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Abstract: Shear test results for a composite wall panel in a light composite structure system are compared with test results for shear walls in Japan. The analysis results show that this kind of composite wall panel works very well, and can be regarded as a solid panel. The composite wall panel with a hidden frame is essential for bringing its effect on shear resistance into full play. Comprehensive analysis of the shear-resistant behavior of the composite wall panel suggests that the shear of the composite shear wall panel can be controlled by the cracking strength of the web shearing diagonal crack.

Key words: composite shear wall; shear test; hidden frame; shear design; cracking strength; shearing diagonal crack

1 Introduction

At present, there are two kinds of thermal insulation methods for external walls, i.e., external or internal insulation of the external wall. These two methods have no relation between the thermal-insulating layer and the structural layer of the wall. The thermal-insulating layer is afterwards glued to the structural layer of the wall, which makes construction complicated and the thermal-insulating layer is easily damaged or falls off. It also brings difficulty to the interior and exterior trims and the installation of air conditioners. Results are not yet reported for the thermal-insulating layer inside the structural layer, and the thermal-insulating layer and the structural layer forming an integrated layer. A light composite structural system developed by our research group can achieve the purposes of thermal insulation and sound-proof without complicating the construction procedure, influencing the interior and exterior trims or the installation of air conditioners.

Compared with several kinds of composite plates used as partition walls now, the composite wall panel as a new kind of residential panel greatly improves the wall panel structural style, retaining its characteristic of assembly time manufacturing. The intermediate tie bars of most similar plates have snakelike shapes. Even the intermediate tie bars of a few similar plates having oblique tie bars are only placed in one way. However, the intermediate tie bars of the composite wall panel in the light composite structural system have a two-way oblique tie bar, which greatly enhances the space integrity of the wall panel. In addition, both sides of similar plates are normally made with normal mortar (cement mortar about 20 mm in thickness), but both sides of the composite wall panel in the light composite structural system are normal poured concrete. Finally, similar plates generally used as light partition walls are connected with the frame structure by tie bars, but the steel wire meshes all around the composite wall panel in the light composite structural system are directly anchored to the hidden frame. All these characteristics make the load carrying mechanism of the composite wall panel in the structure completely different from that of light partition wall plates [1−3]. First, the composite wall panel in the light composite structural system works as the bearing wall of the structure, which can bear tremendous horizontal and vertical loads. In addition, the composite wall panel and its hidden frame work in coordination very well to bear external loads together [4−6].

The light composite structure system is aimed at replacing masonry structures commonly used for multistory buildings in China. The composite wall panels are core structures in this system. In these wall panels, the steel wire meshes in three-dimensional space are the skeleton; inside the panel there is 50-mm-thick thermal insulation and sound-proofing material (polystyrene

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foam plate); on each side of the polystyrene foam plate, there is poured concrete of 60 mm thickness, and both sides of the concrete are connected by space steel wires, so they work together to bear the shearing force, most of the vertical load and the moments. Most of the whole moment and the vertical load is borne by the hidden frame in the system and the wall panel is constrained, so the bearing capacity of the wall panel increases.

To research the behavior of the composite wall panel, various composite shear walls were tested. In this work, the shear test results are compared. The results suggest that the shear design of the composite shear wall panel is controlled by the cracking strength of a web shearing diagonal crack.

2 Test specimens and materials

The specimens for the tests included 13 pieces of wall panels in seven groups. These specimens were identified as SW1 to SW7, and in each group, the specimens were distinguished by SW#-1 and SW#-2. The embedded columns of specimens SW1-1, SW1-2, SW2-1, SW2-2, SW4, SW6-1, SW6-2, SW7-1, and SW7-2 had rectangular sections, and the embedded columns of specimens SW3-1, SW3-2 had T sections; while specimens SW5-1 and SW5-2 had no embedded columns. For specimens SW2, one side was poured mortar 15 mm thick, the other side was poured concrete 45 mm thick. For the other specimens, both sides were poured concrete 30 mm thick, while specimens SW6 was assembled from concrete slabs and SW4 had no polystyrene foam plate and no space connecting steel wire (this group had only one specimen). Specimens SW7-1 and SW7-2 were tested with 294 kN and 196 kN vertical loads, respectively.

The strength grade of the concrete was C20 in the wall slab and the hidden frame and C30 in the foundation. The size of the specimens and the grades and placings of the steel bars are shown in Fig. 1 and Table 1.

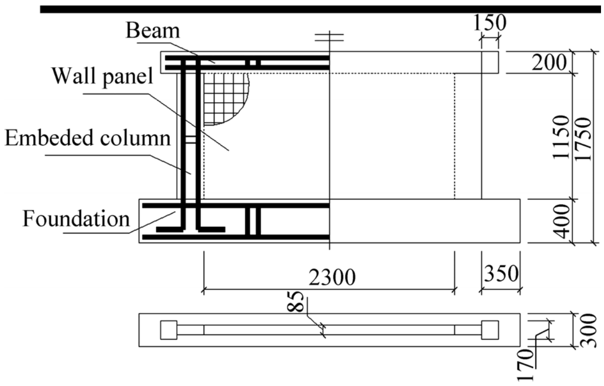


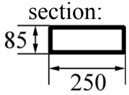
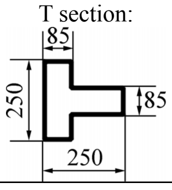
Fig. 1 Test specimen sketch map (Unit: mm)

3 Test results and comparative analysis

3.1 Loading program

Horizontal force was applied by the cantilever loading method, and loading set up of specimens SW1 to SW5 and SW7 is shown in Fig. 2(a). The specimens were bolted on a test-bed, and horizontal force was applied by 500 kN tension and compression jack. Loading set up of specimens SW6 is shown in Fig. 2(b).

Table 1 Size of specimens, grade and placing of steel bars

Test specimen	Size of embedded column/mm	Reinforcement			
		Wall panel	Embedded column*	Foundation*	Beam*
SW1, SW2, SW4, SW6, or SW7	Rectangular section:  85 250	SW4: 2.2@50×50 double layer cold-drawn low-carbon wire mesh without space connecting steel wires; Others: 2.2@50×50 double layer space cold-drawn low-carbon wire meshes	Longitudinal bars: 4ϕ10 grade I hot-rolled steel bars; Hoop reinforcement: 4@100 cold-drawn low-carbon wire	Longitudinal bars: 4@16 grade I hot-rolled steel bars in the top and bottom and 210 grade I hot-rolled steel bars in the middle; Hoop reinforcement: 8@100 grade I hot-rolled steel bar	Longitudinal bars: 2ϕ10 grade I hot-rolled steel bars in the top and bottom; Hoop reinforcement: 4@100 cold-drawn low-carbon wire
SW3	T section:  85 250 85 250	Ditto	Longitudinal bars: 8ϕ10 grade I hot-rolled steel bars; Hoop reinforcement: ditto	Ditto	Ditto
SW5	No embedded column	Ditto	Longitudinal bars: 2ϕ10 grade I hot-rolled steel bars welded at both sides of the specimen	Ditto	Ditto

* Reinforcing bar length for the part extending into the frames was 200 mm on the top and bottom of the embedded columns and 200 mm on both sides of the beams and the stirrup spacing was 50 mm; the length was 700 mm on both sides of foundations and the stirrup spacing was 60 mm.

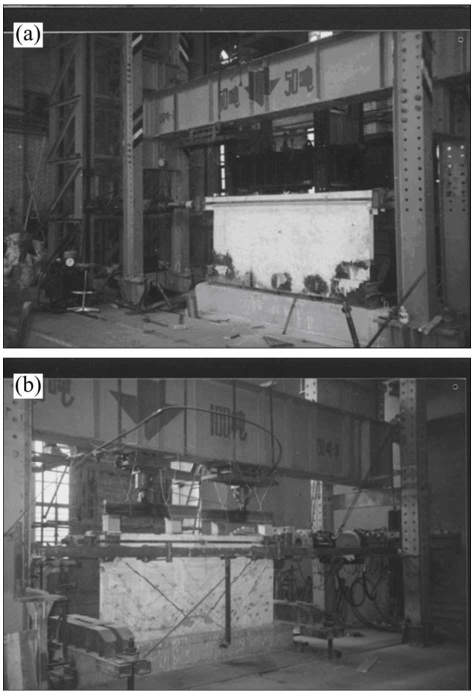


Fig. 2 Loading set up: (a) Front view of loading set up (specimens SW1 to SW5 and SW7); (b) Rear view of loading set up (specimens SW6)

Horizontal force was applied by 1000 kN tension and compression jack, and vertical load which was acted on beam of the specimens in four points by rigid beam was applied by two 500 kN tension and compression jacks, then vertical uniformly distributed load was acted in the middle to lower part of the specimens. In order to ensure applied vertical load with no effect on horizontal deformation of the specimens, the free scrolling hinge support was installed on the top of the jack.

The horizontal loading process was double controlled by load and displacement characteristics, namely load-controlled prior to the specimen yields under multi-stage cyclic load, whereas displacement-controlled after yielding in multiples of the yield displacement value. Each stage used as the loading cycle was applied one time prior to the specimen yielding and applied three times after yielding. Specimen failure was taken as limit state.

Considering difference between the bearing capacities of the specimens being relatively large, so different stage cyclic loads were used in different groups of specimens. In loading process of specimens SW1 and SW4: when horizontal load was equal to ±58.8 kN and ±117.6 kN, a cycle loading was made each, then load was applied according to stage difference equal to 19.6 kN; in loading process of specimens SW2 and SW7: when horizontal load was equal to ±58.8 kN, a cycle loading was made, then load was applied according to stage difference equal to 19.6 kN in loading process of specimens SW3: when horizontal load was equal to ±58.8 kN and ±117.6 kN, a cycle loading was made each, then load was applied according to stage difference equal to 39.2 kN; in loading process of specimens SW5: when horizontal load was equal to ±39.2 kN and ±78.4 kN, a cycle loading was made each, then load was applied according to stage difference equal to 19.6 kN; in loading process of specimens SW6-1, 2: vertical load was equal to 294.0 kN and 196.0 kN each, and it was applied before the horizontal load applied, their loading process of horizontal load was the same as specimens SW3.

3.2 Measuring scheme and instrument arrangement

The purpose of this test is through the low reversed cyclic loading tests of composite wall panel, to gain the stiffness, bearing capacity, deformation of composite wall panel, the stress distribution inside the wall panel, and the failure pattern of the specimens, etc. According to the test purpose, measuring scheme and instrument arrangement are as follows.

1) Strain of steel bar: Strain of steel bar was measured by means of resistance strain gauge. Strain gauges were placed on longitudinal reinforcement of hidden frame, stirrup and wire meshes of the specimens to measure the distribution of internal force of the specimens. Placing of strain gauges is shown in Fig. 3.

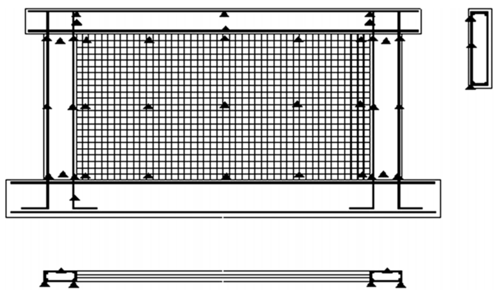


Fig. 3 Placing of strain gauges

2) Lateral displacement at top of the specimens: Lateral displacement at top of the specimens was measured by electronic displacement meter, and the displacement meters were installed at two points namely bottom of upper beam and top of foundation (to measure the slip of whole specimen). To measure additionally shear deformation and rotation angle at bottom of the specimens, five electronic dial indicators were installed, as shown in Fig. 4.

3) Strain of wall surface concrete: Considering that concrete cover being relatively thin and wire meshes and concrete had well bond action and no slip before concrete of the specimens cracking, it can be approximately thought that strain value of wire meshes is the same as that of the concrete at that point. Therefore, only a few 10 mm resistance strain gauges were placed,

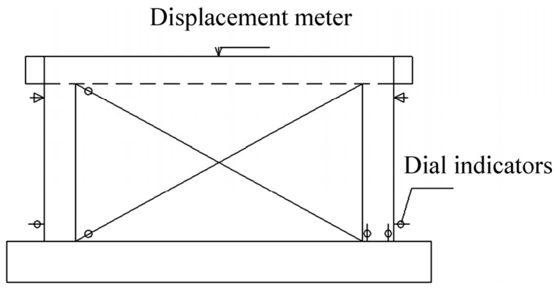


Fig. 4 Installation of displacement meters and dial indicators

to capture concrete cracking. After concrete cracking, dial indicators with long gauge were together used to measure the strain of concrete.

4) Horizontal force: There was a load sensor which can measure load (horizontal force) values at end of loading jack. Load (horizontal force) values and displacement values which were measured by electronic displacement meter can be recorded and draw hysteretic curves through a x−y function recorder, to judge yield and loading control of the specimens. Data gathering used 7V07 intelligent data acquisition instrument, and data of all measuring points under every stage cyclic load were recorded and processed through computer.

3.3 Comparison between composite wall panel with hidden frame and solid wall panel

Test results in Table 2 show that the cracking load of the composite wall panels SW1 is larger than that of the solid wall panels SW4; the ultimate load of the composite wall panel is close to that of the solid wall panel; the two pieces of concrete in the composite wall panel connected by the space steel wire mesh skeleton can work in coordination very well and are similar in shear behavior. The reason for these conclusions is that the rigidity of the vertical plane of the composite wall panel is larger than that of the solid wall panel.

3.4 Comparison of various hidden frames

The test results in Table 2 show that the resistance capacity to a horizontally applied force of the test specimens without embedded columns or with smaller embedded columns is bad. The reason is that without the embedded columns (without longitudinal bars as well) or with smaller embedded columns (with less longitudinal bar as well), there is no constraining or less constraining in the composite wall panel and. Therefore, the shear capacity of the composite wall panel is reduced, which will easily lead to shear sliding failure, destroying the composite wall panel coordination and eliminating the

Table 2 Test results of specimens SW1 to SW7

Style of wall panel	Style of embedded column	$f_{cu,k}$/MPa	Cracking			Ultimate	
			Horizontal force/kN	τ/MPa	$\tau/f_{cu,k}$	Horizontal force/kN	Failure mode
SW1 (composite wall panel)	Rectangular section	22.6	165.6	0.905	0.040	345.6	Failure in bending
			162.3	0.887	0.039	340.5	
		25.1	182.9	1.000	0.040	326.7	
			179.0	0.978	0.039	367.0	
SW2 (composite wall Panel)	Rectangular section	29.1	80.8	0.575	0.020	280.3	Failure in shearing and sliding
			84.8	0.604	0.021	285.8	
			81.1	0.578	0.020	323.3	
			79.8	0.568	0.020	317.7	
SW3 (composite wall panel)	T section	28.9	283.0	1.546	0.052	571.3	Failure in diagonal pressing
			277.9	1.519	0.051	565.2	
		23.9	161.4	0.882	0.037	Failure*	
			198.8	1.086	0.045	567.1	
SW4 (solid wall panel)	Rectangular section	28.9	122.2	0.668	0.023	359.1	Failure in shearing and sliding
			118.9	0.650	0.022	370.2	
SW5 (composite wall panel)	No embedded column	22.0	63.0	0.457	0.016	258.2	Failure in diagonal pulling
		29.5	65.0	0.471	0.029	280.0	
SW6 (assembled composite wall panel)	Rectangular section	23.7	123.9	0.677	0.029	258.2	Failure in shearing and sliding
			120.5	0.658	0.027	280.0	
			101.2	0.553	0.023	275.2	
			98.4	0.538	0.022	265.6	
SW6-1-J	Rectangular section	23.7	—	—	—	382.8	Failure in bending
			—	—	—	357.3	
SW7 (composite wall panel)	Rectangular section	33.1	339.1	1.853	0.056	668.7	Failure in diagonal pressing
			345.0	1.885	0.057	Failure*	
			280.9	1.536	0.046	527.9	
			280.3	1.536	0.046	527.9	

* Failure at loading end.

advantage of the composite wall panel. Without longitudinal bars or with less longitudinal bars, bending horizontal cracks are formed easily, which leads to failure in bending of test specimens. The composite wall panel, with a T section column which is bigger and has more longitudinal bars, has a ratio of the mean shear stress to the concrete cube compressive strength ($\tau/f_{\rm cu}$) of 0.05 (τ is the ratio of horizontal force and cross sectional-area of the specimens), which shows its good shear resistance. Therefore, its shear strength is larger.

3.5 Comparison between composite wall panels with equally thick concrete plates and with different thickness plates

Test results in Table 2 show that the cracking horizontal load of the composite wall panels with unequal thickness concrete plates on the two sides is only 49% of that of the composite wall panels with equally thick concrete plates on both sides and the ultimate bearing capacity of the former is 86% of the latter. For test specimens SW1 (equally thick concrete plates on both sides) and SW2 (unequal thickness concrete plates on the two sides), the other parts were all the same. Though the cube compressive strength of specimens SW2 is higher than that of specimens SW1, the test results show that the stress behavior of SW2 is inferior to that of SW1 for the following reasons. 1) When the thicknesses of concrete plates are not equal, the rigidities of the two sides of the composite wall panel are not the same, so the two sides cannot work together as well and the composite wall panel shear strength is not as high. 2) The thinner side of the composite wall panel with unequal thickness concrete plates cannot tightly anchor on the beam. Therefore, it will slide horizontally when sheared. 3) For the thicker side of the composite wall panels with unequal thickness concrete plates, the reinforcement ratio of the concrete plate itself is relatively low and its rigidity is relatively large, so it has more loading and concrete cracks are easily produced and extended.

3.6 Comparison between composite wall panels with and without vertical pressure

Test specimens SW7-1 and SW7-2 had 294 kN and 196 kN vertical forces applied on the composite wall slabs at four points.

The test results in Table 2 show that the vertical force can obviously increase the shear bearing capacity of the composite wall panel. For the composite wall panel with the 294 kN vertical force, its shear bearing capacity is almost twice as much as that with no vertical force. For the composite wall panel with the 196 kN vertical force, its shear bearing capacity is almost 1.7 times as much as that with no vertical force. Failure of test specimens SW7-1 and SW7-2 was due to diagonal pressing. Diagonal cracks occurred later with no horizontal cracks or sliding produced. These favorable effects of the vertical stress, however, will lead to the danger of vertical tension.

3.7 Comparison between precast and cast-in-place test specimens

Test results in Table 2 show that in a precast composite wall panel assembled using reinforced concrete pins (Fig. 5), the bottom of the composite wall panel can easily incur horizontal cracks and sliding. Therefore, the shear-bearing capacity of the composite wall panel is reduced. This kind of reinforced concrete pin assembling is not reasonable.

To make a thorough inquiry about how to improve the assembly connection, for test specimen SW6-1, the whole test process was divided into two phases. The first phase was the same as for the other test specimens, but with all the concrete pins reinforced by epoxy cement mortar before the tests. The test was conducted until the load reaches the ultimate strength of the test specimen. Before the second phase, controlling braces that restrict sliding were added (Fig. 6), and then the second phase of the test was conducted. The load was applied repeatedly until the test specimen failed. The test specimen SW6-1 for the first phase of the test is denoted by SW6-1, and the test specimen SW6-1 for the second phase is denoted by SW6-1-J. The test results are also given in Table 2, which show that the brace kept the test specimen as one piece. Therefore, this test specimen had a much larger ultimate shear strength.

4 Comparison with shear wall test in Japan

Japanese research on structural earthquake resistance leads the world. Test failure forms of the wall connected by anchor bars in Japan are similar to those in these tests. So in this work, the testing data of each specimen will be compared with the data from Ref. [7].

In Japan, wall shear stresses of wall structure buildings are controlled by producing web shearing diagonal cracks. The mean shear stress is equal to the horizontal load which produces a web shearing diagonal crack in the wall, divided by the cross-sectional area of the wall, and the ratio of the mean shear stress to the concrete cube compressive strength is called the mean shear stress ratio. In Japan, the ratio limit is 0.04 [7]. In our tests of the composite shear wall panel in the system, the ratios for specimens SW1, SW3 and SW7 reached or surpassed this limit.

The shear wall thickness specified in the Japanese code is 150 mm to 180 mm [7]. The thickness of the prototype composite shear wall panel in this test was 170 mm. The reinforcement ratio of shear walls in the Japanese code is 0.15%−0.25%. Reinforcement ratio of

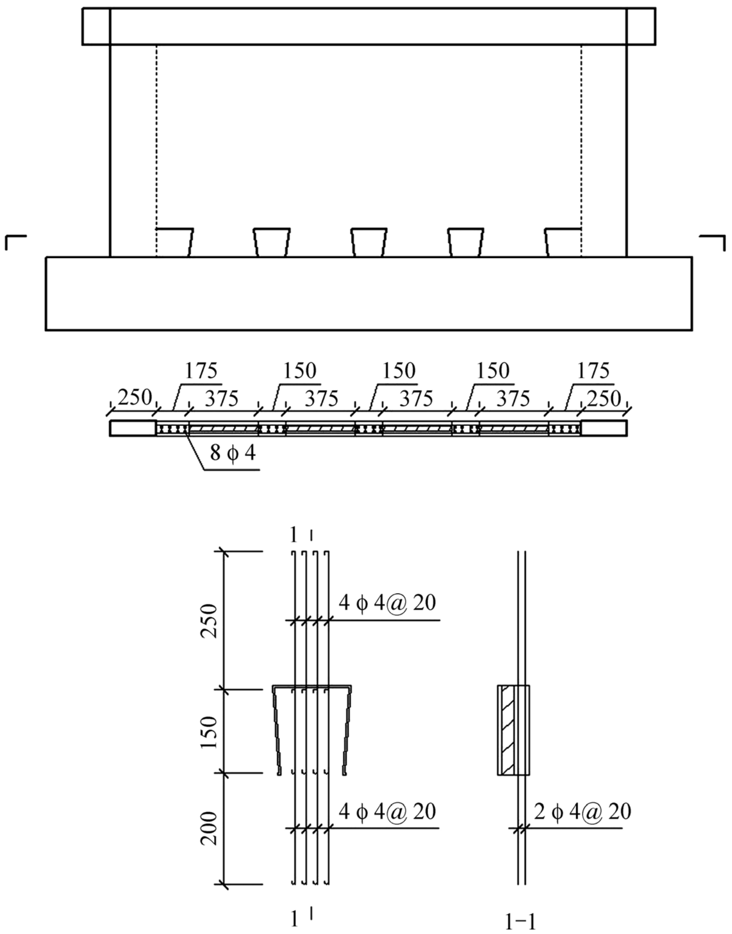


Fig. 5 Placing of pins and their steel bars (Unit: mm)

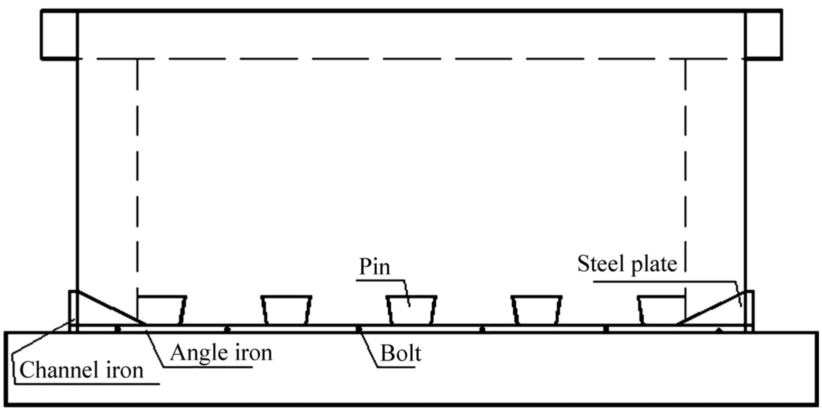


Fig. 6 Sketch map of construction which restricts sliding

the wall in this test was 0.25%.

5 Designing equation for shear bearing capacities of composite shear wall panel

The results show that the ductility of composite walls in a light composite structure system is relatively low. It is recommended, therefore, that shear design of the composite shear wall panel should be controlled by the cracking strength of the web shearing diagonal crack. The equation for calculating the cracking strength was developed as follows.

According to Ref. [8], the cracking strength of a web shearing diagonal crack is mainly related to the reinforcement ratio of the wall panel, the concrete compressive strength, the ratio of wall thickness to embedded column width, the wall cross-sectional area and the vertical compression stress. The theoretical equation for the cracking strength of a web shearing diagonal crack has the form of the Japanese equation [8].

$$cQ_{\mathrm{sc}} = K_{\mathrm{tB}} tD\sqrt{c\sigma_{\mathrm{t}}^2 + c\sigma_{\mathrm{t}}\sigma_0} \tag{1}$$

where $c\sigma_t = 2.2\sqrt{F_c} - 0.0885(F_c)^{1.434}\rho_w^{0.156}$; F_c is the concrete compressive strength and ρ_w is the reinforcement ratio of the wall panel.

$$K_{tB} = \frac{1.0 + 1.05t/B}{1.0 + 1.77t/B} \quad (2)$$

where t is the wall thickness; B is the embedded column width; D is the height of the wall cross-section.

$$\sigma_0 = \frac{9.8N}{A} \quad (3)$$

where N is the vertical compression force and A is the wall cross-sectional area.

The equation for calculating $c\sigma_t$ in the Japanese equation is complex and the coefficient of the equation is dimensional, which is not reasonable. Therefore, the equation, for calculating $c\sigma_t$ should be improved. Let $c\sigma_t = \gamma f_c \rho_{vp}$ and $\sigma_0 = 0.5\ N/A$ and also substitute the symbols in the Chinese code [9] for the corresponding symbols in the Japanese equation. Then, through regression analysis of the test data, the theoretical equation for the cracking strength of a web shearing diagonal crack is

$$V_{cr} = Kb_w h_w \sqrt{\gamma f_c \rho_{vp}\left(\gamma f_c \rho_{vp} + 0.5\frac{N}{A}\right)} \quad (4)$$

where $K = \dfrac{1.0 + 1.05b_w/b}{1.0 + 1.77b_w/b}$; b_w is the wall thickness; b is the embedded column width; h_w is the height of the wall cross-section; f_c is the concrete compressive strength; ρ_{vp} is the reinforcement ratio of the wall panel; N is the vertical compression force; A is the wall cross-sectional area; γ is the coefficient, γ=0.45.

Using Eq. (4), the calculated cracking strength of a web shearing diagonal crack (i.e., horizontal load), for all specimens are given in Table 3. Comparison between the calculated values and the test results shows that except for specimens SW2 and SW4, Eq. (4) is quite appropriate.

Table 3 Cracking load of web shearing diagonal crack

Test specimen	$f_{cu,k}$/MPa	Cracking load/kN	
		Tested	Calculated
SW1	22.6	225.4	226.6
	25.1	264.6	251.7
SW2	29.1	151.9	291.8
		171.5	291.8
SW3	29.8	343.0	342.0
	23.9	328.3	274.3
SW4	28.9	176.4	213.9
SW5	22.0	108.7	—
	29.5	94.8	—
SW6	23.7	230.5	237.6
		205.8	237.6
SW7	33.1	387.1	383.4
		333.2	367.0

The reason why the calculated value of specimens SW2 is much higher is its poor working coordination. For specimen SW4, the rigidity out of the vertical plane is much smaller than that of specimens SW1.

In addition, the design of hidden frames or embedded columns is controlled by the bending cracking strength [10]. To ensure sufficient ultimate bearing capacities of the composite wall panel design, the ultimate bearing capacities [11−15] should also be analyzed.

6 Conclusions

1) The composite wall panel, two pieces of concrete plates connected by a space steel wire mesh skeleton, can bear large loads and work in coordination very well. It can be analyzed as a solid wall panel.

2) Embedded columns and tie beams should be employed in the composite wall panel, so that a composite shear wall structure system with a hidden frame is formed to fully use the shear strength of the composite shear wall.

3) The mean shear stress ratio ($\tau/f_{cu,k}$) for the composite wall panel should be limited to 0.04.

4) In the design of the composite wall panel, diagonal pulling or sliding failure should not occur. Shear design of the composite shear wall panel should be controlled by the cracking strength of the web shearing diagonal crack.

5) For the composite wall panel with equally thick concrete plates on both sides, the load bearing behavior (including crack resistance and ultimate bearing capacities) is very good.

6) The assembled composite wall panel should be designed as an assembled monolithic structure that can resist sliding and local failure.

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