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Shear behavior of precast concrete wall structure based on two-way hollow-core precast panels

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ABSTRACT

In this study, a new precast concrete shear wall system based on two-way hollow-core precast panels (TWHCPP) is proposed. Considering that the ductility of a shear wall is strongly related to its shear behavior, the shear behavior of the newly proposed system was investigated. First, pseudo-static tests were performed for one cast-in-place shear wall specimen and three TWHCPP shear wall specimens. Test results indicate that the cast-in-place shear wall specimen experienced diagonal tension failure and its axial load carrying capacity decreased abruptly when the peak strength was attained. In contrast, the TWHCPP shear wall can effectively avoid such brittle failure and can sustain the vertical and lateral loads at the limit state. Second, a numerical simulation method for the TWHCPP shear wall was proposed and applied to the three TWHCPP shear wall specimens. The predicted strengths and failure mechanisms show good agreement with the experimental results. Finally, a shear strength estimation method was proposed for the TWHCPP shear wall system. Compared to the experimental results, the method yields conservative estimations; thus, it can be a suitable approach to the TWHCPP shear wall design.

1. Introduction

The hollow-core precast panel (HCPP) is one of the most popular precast units in the prefabricated building industry owing to its good structural performance and potential to be mass produced. HCPPs are extensively used as slabs in prefabricated buildings [1]. Because HCPP is easy to fabricate, it has a high degree of standardization, can be mass produced, has a reliable quality, and the hollow core can be used to arrange the distributed reinforcements. It can also be used as a precast shear wall unit in shear wall structures [2,3].

The structural performances of HCPPs as slabs have been studied extensively [4–7], while related research on the use of HCPPs as shear wall units is limited. With reference to existing research studies, Shubow [2] proposed a new precast building system using HCPPs. In this system, HCPPs were used as both slabs and shear wall components, and all components were connected through reinforcements and cast-in-place concretes in hollow cores. Shubow [2] demonstrated the configuration of the new precast building system, but its structural performance was not discussed. Hamid and Mander [3] proposed a shear wall system that used HCPP in combination with unbonded vertical tendons. The shear wall system consisted of multiple HCPP units. The HCPP units at the two ends were tied to the foundation via unbonded vertical tendons, whereas the other units were connected to each other with steel channels, silicone sealant, and rubber block spacers. The specimens were tested by quasi-static loading, and the structure yielded good integrity.

Because the HCPP shear wall system of Hamid and Mander [3] does not have transverse reinforcement and boundary elements, the system is not applicable for high-rise buildings [3]. Capitalizing on the advantages of HCPPs to tackle the aforementioned problems, a new precast shear wall system based on two-way hollow-core precast panels (TWHCPPs) is proposed. Compared with HCPP, TWHCPP is lighter and more convenient to transport. Moreover, transverse and longitudinal reinforcements can be easily arranged in the two-way hollow cores; thus, shear walls with different distributed reinforcement ratios can adopt the same two-way hollow-core panels.

The configuration of a TWHCPP shear wall is shown in Fig. 1. Cast-in-place reinforced concrete boundary elements can be placed at each end of the TWHCPP, that helps to improve the ductility of the system (Fig. 1).

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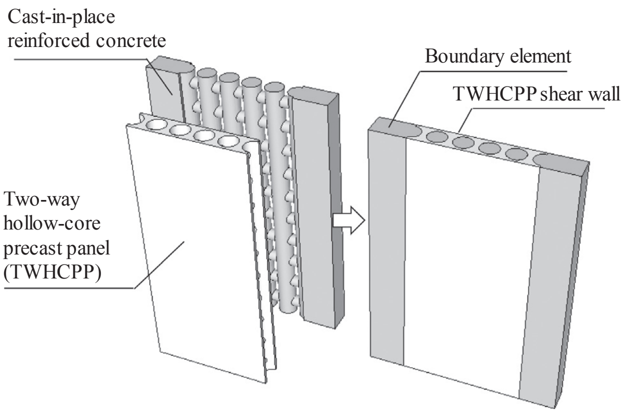

Fig. 1. Schematic of the proposed two-way hollow-core precast panel (TWHCPP) shear wall.

Shear failures (e.g., diagonal tension failure) generally result in brittle failure of a shear wall system. Once they occur, the lateral and vertical load carrying capacities drop abruptly. Therefore, in designing reinforced concrete shear walls, shear strength should be guaranteed to avoid brittle shear failure [8,9]. The shear failure mechanism of cast-in-place reinforced concrete walls has been investigated extensively [10–12]. However, the shear behaviors of the newly proposed precast shear wall system using TWHCPP units are different from the behaviors of reinforced concrete shear walls and should be carefully studied.

Therefore, in this study, the shear behaviors of the newly proposed TWHCPP shear wall are investigated. First, one cast-in-place shear wall specimen (SW0) and three TWHCPP shear wall specimens (SW1–3) with different shear span–depth ratios are tested, and the failure mechanisms of these two types of shear wall units are compared. Second, a numerical simulation method for the TWHCPP shear wall is proposed and then validated by comparisons with experimental results. Third, a shear strength estimation method is proposed for the TWHCPP shear wall system, which can assist in the design of this type of structures.

2. Test specimens and setup

2.1. Test units

The dimensions of the four specimens (SW0–3) are shown in Fig. 2 and Table 1. SW0 is the cast-in-place benchmark specimen. SW1–3 are the TWHCPP shear wall specimens with different shear span–depth ratios in which SW1 has the same dimensions as the benchmark specimen SW0. The axial force ratios of all specimens are set as 0.15. Note that, the axial force ratio is also a very important factor that influences the failure pattern of the proposed precast structure and such effect needs to be carefully investigated in the future.

The reinforcement layouts of all specimens are shown in Fig. 3. Note that specimens SW1–3 are made from the same batch of TWHCPPs. The diameters of the vertical and horizontal hollow cores of the TWHCPPs are 140 mm and 89 mm, respectively, and the distance between adjacent vertical and horizontal hollow cores are 40 mm and 111 mm, respectively.

Because the objective of this study is to investigate the shear behavior of the newly proposed TWHCPP shear wall, all the specimens were designed based on the strong flexural strength/weak shear strength criterion. As shown in Fig. 3, the transverse and longitudinal reinforcements in the TWHCPP are ϕ 8@200 and ϕ 8@180, respectively.

The construction of the TWHCPP shear wall specimens starts from the base block. The reinforcements of the boundary elements and dowel bars were embedded in the base block, as shown in Fig. 4a. The dimensions of the dowel bars are ϕ 12 and 410 mm extended out of the

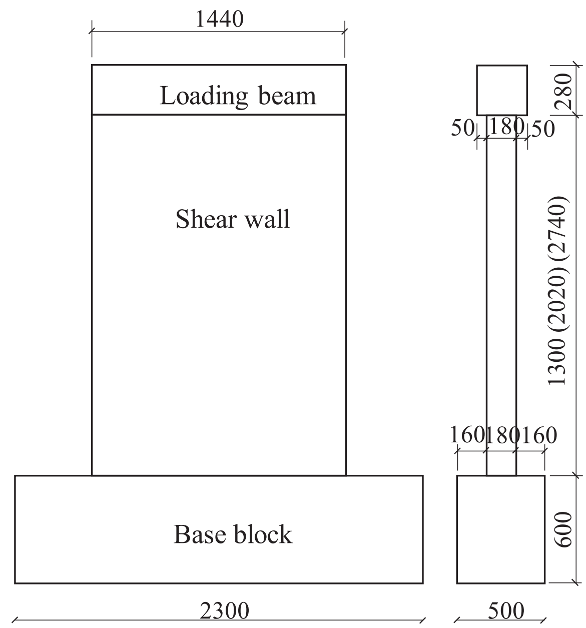

Fig. 2. Dimensions of specimens SW0–3.

Table 1
Parameters of specimens SW0–3.

Specimen	Section dimension (mm)	Height (mm)	Shear span–depth ratio	Axial force ratio
SW0	1440 × 180	2160	1.5	0.15
SW1	1440 × 180	2160	1.5	0.15
SW2	1440 × 180	1440	1.0	0.15
SW3	1440 × 180	2880	2.0	0.15

base block. Once the concrete of the base block reached its expected strength, the TWHCPP unit was installed, and the concrete casting program was then performed for the boundary elements and hollow cores. To ensure connectivity between the boundary elements and TWHCPP, transverse reinforcements of the TWHCPP stretch into the boundary elements, as shown in Figs. 3 and 4. The strength of concretes and reinforcements are listed in Tables 2 and 3.

2.2. Test setup and loading scheme

The specimens were tested under constant axial compression and cyclic lateral load. The test setup is shown in Fig. 5a. The constant axial compressive loading was carried out using a vertical actuator connected to the reaction beam with sliding support. Fifteen strain gauges were applied on the horizontally distributed reinforcements of each panel (MR1–15) (Fig. 5b). Because vertical cracks may form along hollow cores (Fig. 10a), some displacement meters were installed to monitor the slip and gap development along the crack interface. As shown in Fig. 5b, the gap meters (MH2–6) are used to measure the opening at the major vertical crack interface, whereas the slip meters (MV1–6) are used to measure the slip in the major vertical crack interface. The cyclic lateral load was applied by a horizontal actuator installed with the reaction wall. The force–displacement controlled loading program was adopted for the cyclic lateral loading. The early stage of the loading was based on force control. The lateral loads at different cycles were continually increased using a force step F_0. Considering that the strengths of the specimens SW0–3 are different, the force steps were respectively set as 200 kN, 250 kN, and 300 kN, for the specimens with heights of 2.88 m, 2.16 m, and 1.44 m. At each force level, the specimens were subjected to one full cycle. After occurrence of diagonal shear cracks in the specimens, the displacement of this moment was chosen as the