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Seismic performance of precast composite shear walls reinforced by concrete-filled steel tubes

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ABSTRACT

A type of precast slender composite shear wall was proposed and experimentally studied. In this new precast structural wall system, concrete-filled steel tubes (CFSTs) were used to entirely replace the longitudinal reinforcement in the boundary elements of conventional reinforced concrete (RC) shear walls. At joints, the CFSTs and wall web reinforcement were connected by sleeves filled with high-strength mortar. To examine the seismic performance of the proposed system, seven 1/3-scale specimens were built and tested under quasi-static and dynamic cyclic lateral loading with a top displacement rate up to 20 mm/s. Major test variables included axial force ratio ranging from 0.075 to 0.19 and loading rate. This paper reports the damage pattern, hysteretic load-deformation response, energy dissipation capacity, and connection performance of the test specimens. Under the considered axial force levels and loading rates, lateral loads were successfully resisted at the joints and the response of all specimens was dominated by flexure. The use of CFSTs increased lateral strength and deformation capacity. The highest axial force ratio caused drift capacity to be reduced from 2.5% to 2.0%. Although loading rate nearly had no influence on either lateral stiffness or strength, it reduced energy dissipation capacity. Finally, the effectiveness of proposed detailing of sleeve-mortar connections in load transfer was validated by the similar hysteric response, joint opening, and wall sliding between monolithic and precast CFST wall specimens.

1. Introduction

Compared with cast-in-place reinforced concrete (RC) structures, precast RC structural systems offer better quality control, higher construction speed, and reduced construction cost [1]. The strong ground motions in the past three decades examined the seismic performance of precast constructions. The 1994 Northridge earthquake caused severe damage and even collapse in some precast park garage structures. The failures occurred in the gravity systems and the connector elements transferring floor inertia force to lateral systems; properly detailed vertical elements of the lateral systems, such as shear walls or columns, performed well [2]. Due to the deficiencies in design and detailing of roof-to-beam, beam-to-column, and panel-to-structure connections, precast one-story industrial structures suffered severe damage during the 2012 Emilia earthquake in Italy [3–5]. Efforts have been made recently to further study joints and connectors because of their crucial role in the seismic performance of precast systems [6–8]. It is noteworthy that precast office and residential buildings designed and fabricated in accordance with modern codes performed well during the 1989 Loma Prieta, 1995 Kobe, and 2011 Christchurch earthquakes [9–12].

Due to the large lateral stiffness and strength, RC structural walls have been widely employed as lateral load resisting systems for buildings in seismic regions. In contrast to the sizable research on various monolithic RC structural walls, experimental studies of precast walls were much fewer. The seismic performance of precast walls can be strongly affected by the horizontally and vertically aligned connectors of wall segments [13]. Moreover, precast structural walls fall mainly into two categories, post-tensioned (PT) walls and emulative walls. The seismic performance of PT walls has been experimentally studied under static loading [14–17]. The shaking table test conducted by Gavridou et al. [18] on a multi-story precast building confirmed the desired performance of PT walls. Self-centering capacity, which permits limiting seismic damage, is one of the advantages offered by PT walls; this, however, is achieved at the expense of scarifying energy dissipation capacity. Accordingly, various hybrid PT walls with better energy dissipation provided by either mild steel reinforcement or supplemental devices were proposed and investigated [19–22].

Emulative walls, as another class of precast wall system, are expected to have the same ductile performance as monolithically

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constructed walls and dissipate energy through flexural yielding of wall reinforcement. The key to desirable performance of emulative walls is reliably detailing joints, where gap opening and shear slip are of concerns [23,24]. Different types of dry or wet connections for precast RC walls resisting seismic loads were studied [25–28]. Kang et al. [29] recently proposed a precast emulative wall system where the section area of bonded or unbonded rebars above joint are reduced so that plastic hinge can form inside the wall panel and damage to the panel joint can be prevented. Sleeve connection is a simple and thus common approach for splicing wall reinforcement. But this approach faces two challenges that must be overcome. First, the flexural reinforcement of a slender wall is normally concentrated at the boundary elements and enclosed by a large amount of transverse reinforcement needed to confine core concrete and restrain longitudinal reinforcement from premature buckling. The resulting reinforcement congestion makes it extremely difficult to accommodate sleeve connections in the boundary elements. Second, a precast RC shear wall with sleeve connections may not have sufficient ductility. For instance, the lateral deformation capacity measured by drift ratio of a precast RC wall tested by Peng et al. [25] was considerably reduced from 2.4% achieved in a counterpart monolithic RC wall to 1.7%.

This paper presents the experimental study of a new emulative wall system that incorporates the recent advances in steel reinforced structural walls into precast construction. Embedding steel sections in wall boundaries constitutes a type of composite structural wall, which has the potential of both easing construction and improving seismic performance. Qian et al. [30] proposed to use concrete-filled steel tubes (CFSTs), in addition to longitudinal and transverse reinforcement, in the boundary elements of shear walls. The CFST-reinforced walls, tested under high axial load ratios ranging from 0.33 to 0.48 (evaluated based on cylinder compressive strength of concrete), demonstrated greater deformation capacity than conventional RC walls. The ultimate drift ratios of the specimens ranged from 1.0% to 2.0%. Ji et al. [31] tested shear walls with four different types of steel sections embedded into the boundary elements. Subjected to a high axial load ratio of approximately 0.3, the steel-reinforced composite wall specimens presented higher flexural strength and better lateral drift capacity (ranging from 1.3% to 1.6%) than a companion RC wall. Massone et al. [32] suggested the use of wide flange steel sections to partially replace longitudinal reinforcement in the boundary elements of slender walls. The specimens, simulating this type of composite wall deployed in a 15-story prototype building and tested under axial load ratios of 0.10 and 0.18, reached about 2.5% drift ratio prior to experiencing significant strength degradation.

The precast wall proposed in this study has a rectangular cross section, where one steel tube is situated at each end of the wall and filled with self-consolidating concrete. Different from the monolithic composite walls studied by Qian et al. [30], Ji et al. [31], and Massone et al. [32], the CFSTs are used to completely replace the longitudinal and transverse reinforcement in the boundary elements. The wall horizontal reinforcement is welded with the steel tubes for anchorage, which in turn restrains the steel tubes from premature global buckling after concrete cover spalling. Sleeve-mortar splicing connections are used at joints for both CFSTs and wall web vertical reinforcement. An experimental program consisting of testing moderate-scale specimens was developed to examine the cyclic lateral loading response of the proposed precast composite wall system.

2. Experimental program

2.1. Specimens and test variables

Seven cantilevered wall specimens with identical geometry were built and tested. Table 1 presents a test matrix. Fig. 1 shows specimen dimension and reinforcing details. Each specimen consisted of a capital beam, a wall panel, and a base. The capital beam, used to accommodate externally applied axial and lateral loads in testing, was constructed monolithically with the wall panel. The wall panel had a 2700 mm clear height, a 1200 mm width, and a 150 mm thickness. The vertical distance from the line of lateral load applied at the capital beam to the critical section at the base was $h_w = 2850$ mm, leading to a wall height-to-width aspect ratio of $h_w/l_w = 2.38$. The height and width of the wall panel were approximately at 1/3 scale, whereas the thickness was at 3/4 scale. Specimens WSL-0 and WSL-6, serving as control specimens, represented conventional RC walls and monolithic CFST walls, respectively. Precast CFST walls were simulated by specimens WSL-1 through WSL-5, where the base and wall panel were constructed separately and connected by sleeves.

Table 1
Testing matrix of specimens.

Specimen	Wall type	Construction type	Axial force ratio	Axial load (kN)	Loading rate (mm/s)
WSL-0	Conventional RC wall	Monolithic	0.096	572	0.1
WSL-1	CFST wall	Precast	0.099	612	0.1
WSL-2	CFST wall	Precast	0.082	507	5
WSL-3	CFST wall	Precast	0.080	517	20
WSL-4	CFST wall	Precast	0.13	784	0.1
WSL-5	CFST wall	Precast	0.19	1101	0.1
WSL-6	CFST wall	Monolithic	0.075	442	0.1

The specimens were designed to achieve a flexure-dominant response under lateral loading. The wall webs were identically reinforced by two curtains of steel reinforcement. D10 bars (D denotes diameter) with a clear concrete cover of 14 mm were used for web horizontal and vertical reinforcement at a spacing of 150 mm and 200 mm, respectively. The corresponding reinforcement ratios were 0.70% and 0.52%. Each boundary element of specimen WSL-0 consisted of six D16 longitudinal reinforcement, centering at 80 mm from the nearest wall edge, and D8 rectilinear hoops at a spacing of 100 mm for confinement. CFSTs, also centering at 80 mm from wall edge, were used in the boundary elements of all other specimens. Each steel tube had an outer diameter of 102 mm and a wall thickness of 5 mm. Table 2 gives the yield strength, tensile strength, and ultimate strain of the steel reinforcing bars and tubes. Fig. 2 shows the measured tensile stress–strain relationship of reinforcement and steel tube contributing to the flexural capacity of the specimens. Because no clearly defined yield plateau existed for the D10 bars, their yield stress was defined using 0.2% offset method. The tensile yield force of a single steel tube was 86% of that provided by the six longitudinal bars at a boundary element of specimen WSL-0. Shear keys, taking the form of arc reinforcing bar segments, were welded to the steel tubes at an uniform spacing of 300 mm to enhance their deformation compatibility with the surrounding concrete. Each shear key was 100 mm long and had a diameter of 16 mm. The horizontal bars of wall web were weld to the steel tubes. In the precast specimens WSL-1 to WSL-5, the steel tubes and wall reinforcement extended 370 mm and 160 mm below the precast wall, respectively (Fig. 1(c)). These extended lengths, as well as the detailing of sleeve-mortar connections described later, were determined from a series of preliminary monotonic tensile tests for bond strength. In the extruded portion of steel tube, 2.5 mm circular stack welding at a spacing of 20 mm was made on the outer surface to increase its bond with the mortar filled into the sleeve connection.

The base of each specimen was heavily reinforced by D25 longitudinal bars and D10 transverse reinforcement. Fig. 3 shows connection details for the precast specimens. As shown by the dashed lines in Fig. 1(b), steel tube sleeves were embedded into the base to accommodate the steel tubes and wall reinforcement extended from the precast wall panels. The sleeves for wall reinforcement were 120 mm long and had an outer diameter of 32 mm and a wall thickness of 3.2 mm. To