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Fire resistance of concrete-filled steel plate composite (CFSPC) walls

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ABSTRACT

Concrete-filled steel plate composite walls (CFSPC walls) have recently emerged as promising alternatives to conventional reinforcement concrete shear walls due to the benefits of effective space use, less construction time/effort, and higher ductility. While the basic load carrying capacity and seismic performance of CFSPC walls have been receiving great attention, little work has been conducted so far on their fire performance. This paper aims to bridge the knowledge gap by performing 12 fire tests on CFSPC walls, where shear stud spacing, wall height, wall thickness, steel ratio, and fire scenario (uniform or single-sided fire) were considered as the main test parameters. For the uniform fire tests, the specimens exhibited reasonably good fire resistance, where the fire resistance time exceeded 2 h. Three dominating failure modes were observed, namely, local buckling of steel plates (LB), severe cracking ('tearing') of the weld along the corner line of the wall (WC), and global instability (GI). For the specimens subjected to single-sided fire, minor local buckling of the heated steel plate accompanied by tie bar failure in the buckled areas were observed, but no further load bearing failure was induced by the end of the heating process. The fire resistance time, which exceeded 2.5 h for the specimens exposed to single-sided fire, was governed by the 'thermal insulation criterion'. After 2.5 h of heating, some specimens lost their thermal insulation capacity. Based on the test phenomena, preliminary design recommendations are proposed, and future research directions are also outlined.

1. Introduction

A shear wall (or bearing wall) is a vertical structural walling system that carries vertical loading and concurrently is capable of resisting lateral loads in the plane of the wall through shear and bending. While conventional reinforced concrete (RC) walls have been widely used in multi-storey buildings and are shown to be effective in resisting both vertical and in-plane horizontal loadings, cracks in the surface of the wall can occur due to seismic action and other environmental effects (e.g., thermal action) [1]. The cracks, when accompanied by spalling and splitting of concrete, can lead to deteriorations of wall stiffness and strength, and the exposure of the reinforcement to air can raise corrosion issues, which further compromises the overall performance of the shear wall. In addition, when the building height is considerable, the increase of the axial compressive force can make the RC shear walls excessively thick, which is not efficient in terms of space use. The RC shear walls are also quite demanding in terms of construction time and effort (e.g., concrete curing time and formwork). These disadvantages can in some cases limit the application of conventional RC shear walls, especially in high-rise/super high-rise buildings, offshore structures, and nuclear power facilities. To overcome these issues, concrete-filled steel plate composite shear wall systems (hereafter named as CFSPC walls) have emerged as promising alternatives to conventional RC shear walls. A piece of CFSPC wall normally consists of two exterior steel plates filled with concrete, where the composite action is typically achieved via steel headed shear studs which are welded to the inner surface of the exterior steel plates. The steel plates serve as reinforcements and permanent formwork for the concrete infill and provide cover for possible cracks in the concrete. The presence of the shear studs and the concrete tends to increase the local buckling resistance of the steel plates, and concurrently the steel plates offer constraining effect for the concrete. Compared with conventional RC walls, CFSPC walls can be thinner and lighter, which also benefits the foundation system.

CFSPC walls or those employing similar working principles (e.g., concrete-filled profiled steel sheeting walls) started to attract attention from more than a decade ago. Wright [2] conducted four pilot tests on axially-loaded concrete-filled profiled steel sheeting walls. It was preliminarily found that the development of full axial capacity could not be achieved due to the low interface bond strength offered by

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Table 1
Details of test specimens and test results.

Fire scenario	Specimens	Height (H)	Width (B)	Wall thickness (T)	Steel plate thickness (t)	Steel ratio ρ_s	Shear studs	Tie bars	Load (kN)	Design load ratio n_d	Fire resistance time t_r (min)	Axial shortening at R-failure (mm)	Failure mode
Uniform fire	SCW1	850	1000	150	3	5.2%	ϕ2@40	ϕ10@160	1500	0.34	161(R)	7.9	LB
	SCW2	850	1000	150	3	5.2%	ϕ2@60	ϕ10@160	1500	0.34	178(R)	8.4	LB
	SCW3	850	1000	150	3	5.2%	ϕ2@80	ϕ10@160	1500	0.34	174(R)	8.5	LB
	SCW4	1350	1000	150	3	5.2%	ϕ2@40	ϕ10@160	1500	0.34	156(R)	11.8	WC
	SCW5	1850	1000	150	3	5.2%	ϕ2@40	ϕ10@160	1500	0.34	133(R)	18.0	GI
	SCW6	850	1000	200	4	5.5%	ϕ2@40	ϕ10@160	2000	0.34	212(R)[a]	8.3	LB
	SCW7	850	1000	150	2	3.4%	ϕ2@40	ϕ10@160	1500	0.40	178(R)	8.5	WC
	SCW8	850	1000	150	5	8.5%	ϕ2@40	ϕ10@160	1500	0.26	166(R)	7.6	LB
Single-sided fire	SCW9	1000	1000	150	3	5.2%	ϕ2@40	ϕ10@160	1500	0.34	191(I)	–	TIF
	SCW10	1000	1000	200	4	5.5%	ϕ2@40	ϕ10@160	2000	0.34	> 207(I)[b]	–	NF
	SCW11	1000	1000	150	5	8.5%	ϕ2@40	ϕ10@160	1500	0.26	185(I)	–	TIF
	SCW12	1000	1000	150	3	5.2%	ϕ2@40	No tie bars	1500	0.34	166(I)	–	TIF

All unit in mm unless stated otherwise.
LB=local buckling, WC=weld cracking, GI=global instability, TIF=thermal insulation failure, NF=no failure.
[a] oil pump replacement time (30 minutes) is deducted from the total fire resistance time.
[b] test terminated early due to malfunction of the furnace ventilation system.

commercially-available profiled decking. A recent study on similar wall types [3,4] revealed the optimum spacing of interface connectors to prevent shear buckling of profiled steel sheets before yielding. Although the focus of the study [3,4] was given to the performance of the walls under monotonic shear, a later study conducted by Zhang et al. [5] confirmed that a reasonable consideration of shear connector spacing could significantly improve the local buckling performance of composite walls. Link and Elwi [6] investigated the transverse and longitudinal loading capacities of CFSPC walls used in offshore structures against ice and wave impacts. Numerical results were also given which were consistent with the experimentally observed behaviour. Ozaki et al. [7] studied the influences of axial force level, partitioning diaphragm and openings on the cyclic performance of CFSPC walls, and design equations were proposed to consider those influential factors. The local buckling behaviour and ultimate strength of the steel plates in CFSPC walls under biaxial compression and in-plane shear was investigated by Liang et al. [8], where the critical stud spacing for mitigating critical local buckling of steel plates was proposed. Zhao and Astaneh-Asl [1] proposed a new composite wall system consisting of a steel plate wall bolted to a RC shear wall with a gap left between the RC wall and the boundary steel frame. It was found that, compared with the case with no such gap, the new system could be more ductile and the concrete had less damage. Cyclic tests were performed by Eom et al. [9] to investigate the seismic behaviour of CFSPC walls with rectangular and T-shaped cross sections. The wall specimens were found to fail mainly by tensile fracture of the welded joints at the wall base and coupling beams, or by local buckling of the steel plates. The ductility of the walls was significantly affected by the strengthening methods used for the wall base. More cyclic tests were later conducted by Nie et al. [10], and it was found that with boundary elements on both sides of the wall, the specimens exhibited good energy dissipation ability and deformation capacity with full hysteretic curves and large ultimate drifts. Based on the experimental work, numerical and analytical investigations were also performed to study the stiffness and deformation capacity of such walls under in-plane shear [11,12].

It can be seen from the above discussion that the existing studies have mainly considered the ambient behaviour of CFSPC walls, whereas little work has been conducted on their fire performance. There is no doubt that the ambient behaviour of CFSPC walls subjected to axial compression and/or in-plane shear are of interest to structural engineers (especially seismic engineers), a good understanding of their fire performance is equally essential. This is because that when employed in building structures, these wall systems are critical components against structural failures, and importantly, they are crucial thermal barriers for preventing or delaying the spread of fire and smoke. For offshore structures and nuclear power facilities, where CFSPC walls are gaining great popularity, the risk of blast and fire is also a matter of great concern. Unlike the conventional RC walls where the reinforcements can be protected by the concrete, the exterior steel plates of CFSPC walls, if unprotected (i.e., no fire-proof painting), will be directly exposed to fire and thus experience rapid deterioration of strength and stiffness. The constrained thermal expansion may further decrease the fire resistance of the wall due to local buckling of the steel plates. Although Varma et al. [13] carried out a numerical study on axially loaded CFSPC walls subjected to increased air temperature by up to 350 °C for up to 3 h and found that no local buckling of the steel plates occurred when the s/t ratio (s=shear connector spacing, t=steel plate thickness) was less than 20, the increase of air temperature can be much higher than that considered in Varma et al. [13] (i.e., 350 °C) under real fire scenarios, and thus unexpected failure may occur.

In order to bridge the above-mentioned knowledge gap, this paper addresses the fundamental fire performance of CFSPC walls. A total of 12 fire tests on CFSPC walls are reported, where the test setup and procedure are described in detail. The test results, such as failure modes, temperature distributions, deformation responses, and fire resistance time, are presented, and the influences of shear stud spacing, wall height, wall thickness, steel ratio, and fire scenario are thoroughly discussed. Based on the test phenomena, preliminary design recommendations are proposed, and future research directions are also outlined.

2. Experimental programme

2.1. Test specimens

A total of 12 specimens (SCW1 to SCW12) were tested in the experimental programme, as shown in Table 1. The considered test parameters included shear stud spacing (s), wall height (H), wall thickness (T), steel ratio (ρ_s), and fire scenario (i.e., uniform fire exposure or non-uniform/single-sided fire exposure). The width (B) of the walls was 1000 mm and three different heights (H), namely 850 mm, 1350 mm, and 1850 mm, were considered for the specimens exposed to uniform fire. For those exposed to single-sided fire, a consistent height of 1000 mm was adopted. The wall thickness (T) was taken as 150 mm or 200 mm, and four steel plate thicknesses (t_s), namely, 2 mm, 3 mm (control case), 4 mm, and 5 mm, were consid-