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Cyclic shear behavior of composite walls with encased steel braces

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ABSTRACT

The composite wall with encased steel braces (ESB wall) is a novel type of steel–concrete composite wall that consists of a steel braced frame embedded in reinforced concrete. This arrangement is supposed to enhance the seismic performance of the wall, as the steel columns encased in the boundary elements can increase the flexural strength of the wall and the steel braces encased in the web can increase the shear strength. This paper examines the cyclic shear behavior of squat ESB walls by conducting quasi-static tests. Two wall specimens with an encased single-diagonal brace, one of which had an I-shaped brace and the other a steel plate brace, were tested. Both specimens had a shear-to-span ratio of 1.06, and they were subjected to a moderate axial force ratio of 0.18. A flanged wall section was intentionally used to ensure that the wall's flexural strength exceeded the shear strength, resulting in a shear failure mode. The two specimens failed in a similar manner, characterized by crisscrossed-diagonal cracking and crushing of the concrete in the web panel. Hysteretic responses of the two specimens were nearly identical, and both types of steel braces buckled after they yielded. This indicates the potential use of steel plate braces instead of I-shaped braces in ESB walls, as the former allows improved efficiency and quality of construction. A simple method was presented to calculate the cracked shear stiffness of ESB walls, and this method could reasonably estimate the cracked shear stiffness of the specimens. In addition, the JGJ 138-2012 formulas for assessing the shear strength of ESB walls were calibrated through the analysis of data collected in past tests and in the present experimental program. The design formulas, based on the superposition method, are found to provide reasonable and conservative predictions of the shear strength capacity of ESB walls.

1. Introduction

As reinforced concrete (RC) walls can provide high lateral stiffness and strength, they are widely used as the major lateral force-resisting components in high-rise buildings. Steel-reinforced concrete (SRC) walls have been developed to further increase the flexural strength and lateral deformation capacities of RC walls, where structural steel is encased in the wall's boundary elements [1–6]. More recently, a novel type of composite wall that includes encased steel braces (ESB) has been proposed for enhanced seismic performance of high-rise building structures. Fig. 1 shows a schematic view of the ESB walls, where the steel braced frame is embedded in the reinforced concrete. The use of diagonal bracing members is supposed to increase the shear strength capacity of the walls.

ESB walls have seen use in super-tall building structures constructed in regions of high seismicity in China. The ESB walls are commonly used on stories where the shear force demand is very high (e.g., the core walls on lower stories or on the outrigger story). In current practice, I-shaped steel is usually adopted as the encased braces. However, the use of I-shaped steel braces could make the casting of concrete difficult. As for structural members subjected to cyclic axial tensile and compressive forces, the steel braces do not allow the members to have vent holes. It is thus difficult to ensure the compactness of the concrete in the region near the intersection of the flanges and the web of the brace. To overcome this problem, one solution may be to use steel plate braces instead of I-shaped steel braces. However, steel plate braces, which are slenderer than I-shaped steel braces, are prone to buckling, after the surrounding concrete sustains damage under cyclic loading and loses the restraint to the encased braces. The present paper reports the quasi-static tests of two squat ESB walls subjected to a moderate axial compressive force and cyclic shear loading. The two ESB walls are designed to have nominally identical shear

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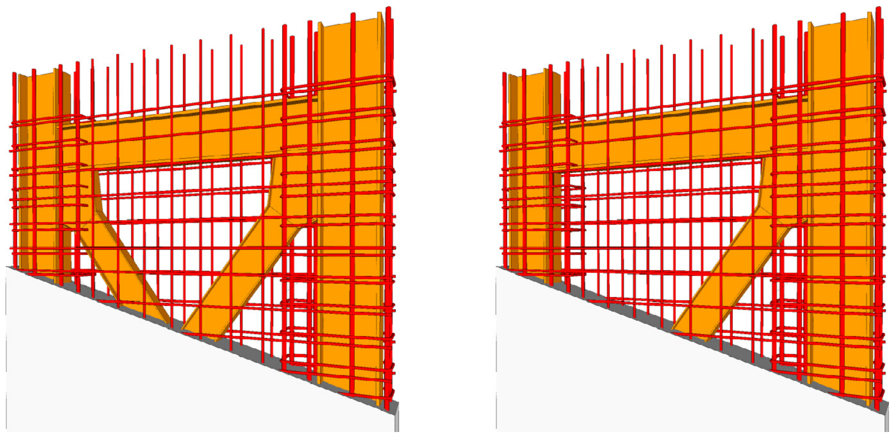

(a) With encased X-shaped braces (b) With an encased single-diagonal brace

Fig. 1. Composite walls with encased steel braces (ESB walls).

strength, although one adopts the I-shaped steel brace and the other the steel plate brace. By comparing the test results, the objective of this paper is to identify the possibility of using steel plate braces in ESB walls.

In the past decade, a few experimental tests [7–10] have been conducted to investigate the seismic performance of ESB walls. Additionally, the Chinese code for design of composite structure (JGJ 138-2012) [11] specifies the design formulas and requirements of ESB walls. Another objective of this paper is to calibrate the JGJ 138-2012 formulas for assessing the shear strength of ESB walls by analyzing data from past tests and the present experimental program.

2. Experimental program

2.1. Test specimens

The prototype wall was selected from the Z15 Tower, a super-tall building of 528 m height, located in the central business district of Beijing. The building uses the core wall–mega braced frame interaction system, as shown in Fig. 2(a). The peak ground acceleration of the design basis earthquake (DBE, with a probability of exceedance of 10% in 50 years) for the site is 0.2 g. The period of first vibration mode of the Z15 Tower is approximately 7.5 s, and the corresponding spectral acceleration at DBE is 0.07 g with an assumed damping ratio of 5%. Because of the large strength demand on the core walls induced by seismic action, steel–concrete composite walls are used for this building. A wall pier (see Fig. 2(b)) located on the 88th story was taken as the prototype in this study. The prototype wall had an encased single-diagonal steel brace.

The wall specimens were scaled down by 0.35 in dimension and by 0.12 in shear strength capacity relative to the prototype walls, to accommodate the capacity of the loading facility. Two wall specimens (labeled SW1 and SW2) were designed, SW1 having an encased steel plate brace and SW2 having an encased I-shaped brace. The two specimens had identical geometry, as shown in Fig. 3. Each wall was 1332 mm tall. A flanged wall section was intentionally used to ensure that the walls fail in shear mode. The web wall majorly resisted the in-plane shear, while two flange walls majorly resisted the overturning moment. The wall section had a cross-sectional depth, flange width, web thickness, and flange thickness of 1470, 300, 140 and 150 mm, respectively. A foundation beam and a top beam were cast together with the wall.

Fig. 4 shows the cross section and reinforcement details of the specimens. The steel rebars used in the two specimens were identical. ϕ8 (diameter of 8 mm) rebars were placed as distributed reinforcement at the web wall of specimens. The horizontally distributed rebars were spaced at 95 mm with a reinforcement ratio of 0.75%, and the vertically distributed rebars were spaced at 110 mm with a reinforcement ratio of 0.65%. Eight ϕ14 (diameter of 14 mm) and four ϕ18 (diameter of 18 mm) rebars were placed at the boundary elements as longitudinal reinforcement, corresponding to a 3.1% reinforcement ratio (i.e., the ratio of the gross cross-sectional area of the longitudinal rebars to that of the boundary element). ϕ8 rebars, in the form of hoops and cross ties, were used as the boundary transverse reinforcement. Some hoops were made of U-shaped rebars with both free ends welded to the web of embedded steel columns. The volumetric ratio ρ_v of the boundary transverse reinforcement was 2.0%. The corresponding mechanical volumetric ratio $\lambda = \rho_v f_{yv}/f_c$ was 0.36, where f_{yv} and f_c denote the yield strength of the boundary transverse reinforcement and the axial compressive strength of the concrete.

The steel columns were made of built-up I-shaped steel and were embedded at the wall boundary elements. The steel columns had a cross-sectional depth of 185 mm, flange width of 90 mm, and flange and web thickness of 10 mm. The width-to-thickness ratio of web h_w/t_w was 16.5, and the width-to-thickness ratio of flange $b/(2t_f)$ was 4.5. The reinforcement ratio of the embedded steel column (i.e., the ratio of the cross-sectional area of embedded steel to that of the boundary element) was 4.7%. The I-shaped steel beams were embedded at floor height for the prototype wall, and they were embedded within the top beams and foundation beams for the specimens. The steel beams were rigidly connected to the columns using fully welded connection details. ϕ8 (diameter of 8 mm, length of 35 mm) shear studs were welded to both flanges of the steel beams and columns to develop the composite action with surrounding concrete.

The steel plate brace used in Specimen SW1 was 100 mm wide and 10 mm thick. The built-up I-shaped steel brace used in Specimen SW2 had a cross-sectional depth of 126 mm, flange width of 42 mm, and flange and web thickness of 5 mm. The cross-sectional areas of the two braces were identical. As shown in Fig. 4, ϕ8 headed studs were welded to the braces to provide the composite action between the steel brace and RC encasement. The single diagonal brace was set with its centroid line passing through the intersection point of the beam and column. The inclination angle of the brace was 52.8°. JGJ 138-2012 [11] specifies a limiting slenderness ratio of $120\sqrt{235/f_y}$ for the encased braces. This limiting value may be overly strict, as it is identical to the limit for the braces in a steel braced frame without additional restraint.