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Experimental work on single and double-sided steel sheathed cold-formed steel shear walls for seismic actions

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ABSTRACT

Experimental investigation is presented on cold-formed steel (CFS) shear walls comprising single and double-sided steel sheathing. Cyclic loading tests were performed on six CFS wall specimens. The observed predominant failure modes include sheathing buckling, sheathing-to-frame connection bearing/tilting and chord stud buckling. The walls developing sheathing connection failure show higher energy dissipation than the walls imposing chord stud buckling. Using double-sided sheathings increases the energy dissipation, shear strength and elastic stiffness by up to 70%, 63% and 115%, respectively compared to those of single-sided sheathed walls. On the use of sheathing on both sides the chord stud failure must be avoided.

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1. Introduction

The use of steel sheathing on CFS stud walls is mainly to achieve higher shear resistance in extreme loading incidents. This is an approved lateral force-resisting system in ASCE7-10 [1] for low-rise buildings. American Iron and Steel Institute (AISI) [2] presents the nominal shear strength of steel sheathed CFS shear walls of 0.457 and 0.686 mm sheathing thicknesses with aspect ratios (i.e. height-to-width ratios) of up to 2:1 and 4:1, respectively. This is based on an experimental investigation conducted by Serrette et al. [3,4]. They showed that in shear walls with widely spaced sheathing fasteners the predominant mode of failure is pull-out of the fasteners associated with a significant out-of-plane deformation in the sheathing. In shear walls with closely spaced fasteners, however, sheathing and stud buckling were dominant.

The rapid growth of CFS structures in the housing market worldwide requires greater range of aspect ratios and sheathing thicknesses for CFS shear walls. Cheng Yu [5] conducted an experimental program to determine the shear strength of the walls having sheathing thicknesses of 0.686, 0.762 and 0.838 mm and aspect ratios of 4:1 and 2:1. Buckling of the sheathing plates and pull-out of sheathing screws were reported and it is concluded that the code reduction factor represents fairly well the strength reduction for aspect ratio of 4:1.

In another effort by Yu [6] physical testing was performed on different wall configurations having 2.44 m height and 1.83 m width. The test results showed that by using blocking and strapping elements lateral buckling of the interior studs can be prevented. This leads to noticeable increase of the shear strength and the ductility capacity. The design values obtained by Yu et al. differ from those presented by Serrette et al.; Ellis [7] conducted a series of shear wall tests and concluded that this difference was mainly due to the use of different cyclic loading protocols.

DaBreo et al. [8] and Balh et al. [9] tested a number of walls with various configurations at McGill University. The walls were differentiated by framing and sheathing thicknesses, screw fastener detailing, aspect ratio and framing reinforcement. They concluded that, in general, the use of closely spaced sheathing fasteners and thicker sheathing panels can lead to a higher shear resistance values. This is provided the stud members are designed to carry the increased shear resistance by means of blockings and the capacity based design approach. The decrease of the spacing of screws does not necessarily result in the increase of the shear resistance. This was revealed in a series of experiments by Javaheri-Tafti et al. [10] on CFS walls sheathed by steel sheets. The frame members were made by light C section with 0.75 mm thickness led to chord stud failure when the screw spacing was decreased.

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The extensive use of CFS shear walls to reach a higher shear resistance in a building, however, limits the architectural flexibility. These immovable walls disturb the open spaces and future planning alterations. A solution to minimize the use of CFS shear walls is to employ double-sided steel sheathing. This is also aesthetically pleasant in respect of the material consistency of using steel for the whole frame and sheathing. Several experimental work [3–10] have been implemented on different configurations of single-sided sheathing; whereas, the use of double-sided sheathing could be more efficient in terms of providing higher shear resistance with less space occupancy throughout. While research has been carrying out on single and double-sided OSB, gypsum and calcium silicate sheathed walls [11–13] lack of research is evident on double-sided steel sheathing. Further, insufficiency of design specification [2] limits the use of double-sided steel sheathed walls. The only specified design expression [2] in the case of using wooden sheathing (or OSB) on one side and fully blocked gypsum board on other side is a 30% increase of the shear strength values. On the use of double-sided steel sheathed walls the axial force demand in chord studs will be increased, thus the need for thicker framing members compared with the common range from 0.75 to 1.372 mm used in the aforementioned studies. The challenging issue is to delay the stud failure, as the main gravity load bearing element, which is more problematic in double-sided sheathed walls.

A comparative experimental study was conducted in this research to investigate the structural performance of single and double-sided steel sheathed walls. Six wall specimen configurations were tested under cyclic loading and the results are presented as follows.

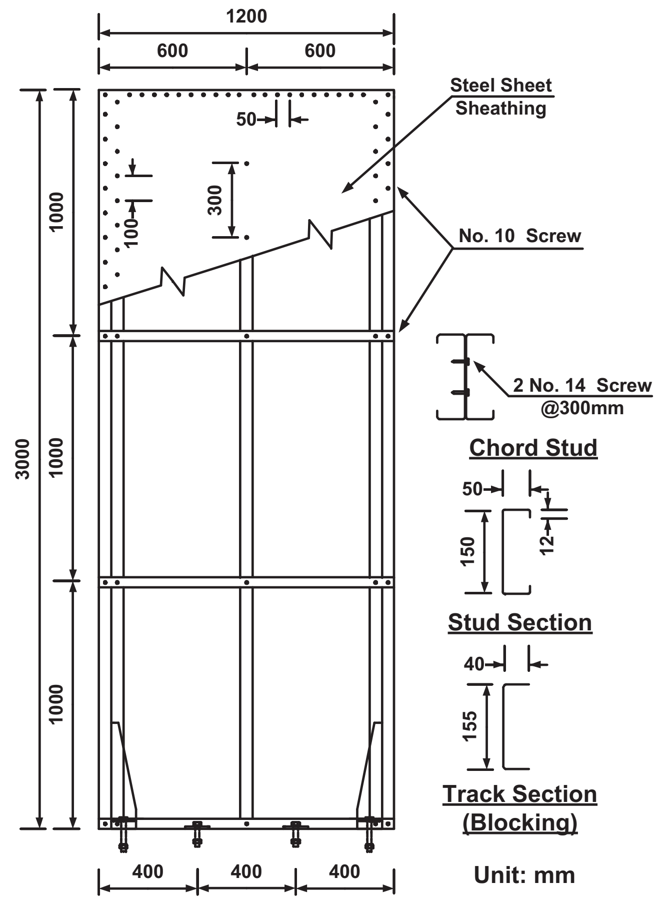

Fig. 1. Framing details and screw arrangement for shear walls.

2. Testing arrangements

Listed in Table 1 are six sheathed wall configurations designed for the experimental investigation. Both single and double-sided sheathings have been employed for comparison purposes with different framing and sheathing thicknesses.

Specimens A1 and A2 were designed to impose a buckling failure in the chord studs which is unfavorable in respect to the design concept that requires delay of the stud failure. Specimens B1 and B2 were designed to fail through the screw connections between the sheathing and the frame. Finally, for Specimens C1 and C2 the failure is expected at the screw connections and the chord studs respectively.

2.1. Specimen details

Fig. 1 shows a typical detailing and screw spacing arrangement for the specimens. The overall dimensions of the specimens were 1.2 m wide and 3.0 m high with studs placed at 0.6 m centerline spaces. Double back-to-back channels were used for the exterior studs, and single studs were positioned between them. Single tracks were used at the top and bottom of the walls. The studs were connected to the top and bottom tracks through the flanges using three No. 10×19 mm self-drilling– self-tapping pan head screws. The nominal depth of the studs and tracks was 150 mm. The webs of the double studs were attached by two lines of No. 14×32 mm hex washer head (HWH) self-drilling screws with 300 mm spacing between the screws in each line. No. 10×19 mm self-drilling – self-tapping pan head screws were used for sheathing to frame connections. The screws were arranged in a single line on the tracks and in a staggered pattern on the chord studs with 50 mm spacing. The latter is to reduce the loading eccentricity on the chord studs as suggested by Yu et al. [5]. The edge distance of the sheathing screws was 20 mm on tracks and 25 mm or 75 mm on chord studs.

In specimens with sheathing on both sides, the second side sheathing was assembled after the specimen installed on the test rig. This was to provide access to the bolts of the hold-down to the base beam connections. Blocking members were placed at one-third and two-thirds of the height of the walls having the same section as the tracks. They were connected to the interior and chord studs as shown in Fig. 2. It was previously shown [6,8,9] that the use of blocking/strapping members can help preventing failure/damage to the interior and chord studs. The blocking connections detailed to provide higher degree of restraining effect to the studs by the use of continuous flanges at both sides of the stud sections.

To resist shear forces four ASTM A325 16 mm diameter bolts (two at each side) were used to connect the bottom track to the base beam. Fig. 3 shows the hold-down dimensions having relatively thick plates to ensure no uplift would occur. To resist the overturning forces, the hold-downs were connected to the base beam by two ASTM A490 20 mm diameter bolts. Each hold-down was attached to the chord stud by three lines of No. 14×32 mm hex washer head (HWH) self-drilling screws with

Table 1
Shear wall test specimens.

Specimen	Nominal framing thickness (mm)	Nominal steel sheet thickness (mm)	Single sided/ Double sided
A1	1.25	0.8	Single sided
A2	1.25	0.8	Double sided
B1	2.50	0.8	Single sided
B2	2.50	0.8	Double sided
C1	1.25	0.6	Single sided
C2	1.25	0.6	Double sided

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