# Experimental investigation into the performance of cold formed steel walls sheathed with OSB and cement based panels

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**Abstract**

Cold-formed steel (CFS) structures are developing fast in both seismic and not seismic areas for their high degree of prefabrication, high structural performance and the good energy performance. In the last decades, many research groups around the world have focused on the analysis of seismic behaviour of CFS structures and the development of design procedures that still need to find their full implementation in many international codes, first of all in the Eurocode. Therefore, the development of CFS system still requires, very often, the adoption of large experimental testing, in particular, when sheathing braced design methodologies are applied. This paper presents, for the first time, the in-plane tests of CFS walls sheathed on one side with oriented strand boards (OSB) and cement based panels (CP). In particular, a set of two full scale tests of ledger walls having 2400mm width and 2926mm height, sheathed with OSB3 and CP, and a set of two tests on ledger walls having dimensions (2400mmm x 2974mm) sheathed with OSB3 panels are presented and discussed. The tests have been carried out in accordance to the BS EN 594:1996. The results confirm in agreement with literature studies that the collapse is governed by the failure of the screws between panels and steel profiles. And, more importantly it shows that, when both OSB and CP are present the collapse is governed by the failure of the screws between cement panels and steel profiles. When comparing the two sets of walls those sheathed only with OSB panels have lateral resistance which is about 1.5 time higher than when also CP are present.

**Keywords**

Cold-formed steel, sheathed shear wall, fastener behaviour, monotonic test, shear strength.

## 1 Introduction

Structural systems based on cold formed steel (CFS) are increasingly adopted in both seismic and non-seismic areas for their capacity to be manufactured at large scale, their modularity, their high structural performance and low environmental impacts [1]. For decades, research groups have advanced knowledge of their structural performance, but the lack of codification of their lateral capacity still undermine their full application in European countries.

In UK, in particular, these systems have the potential to help towards the current housing crisis, since they have the potential to be fabricated and delivered at large scale [2]. Analysis and understanding of the lateral capacity of CFS system is essential to predict their behaviour under wind loads and/ or seismic loads.

CFS systems can be designed according to either an "All steel design" or "Sheathing braced design" approach [3].

When sheathing braced design methodologies are applied then, the racking capacity is achieved through the collaboration between CFS members and sheathing panels, and in particular the connections between them plays a fundamental role. In order to assess the lateral and seismic performance of sheathed cold-formed steel (SCFS) structures, several experimental and/or numerical research programs have been carried out on different wall configurations [4-12}. In the last 30 years, more than 600 tests have been carried out on walls with height ranging between 2.4 m and 3.60 m and width between 1.2 and 7.3 m, and sheathed by cement plaster (CP), chipboard (CHI), fiberbond wallboard (FBW), gypsum sheathing board (GSB), gypsum wallboard (GWB), oriented strand board (OSB), plywood (PLY), steel corrugated sheet (SCS), steel sheet sheathing (SSS), steel flat strap X-bracing (X-B) or calcium silicate board (CSB). Monotonic tests and cyclic tests have been carried out, and some of those tests have informed the development of current design codes as the Uniform Building Code – Edition 1997 (International Conference of Building Officials. Whittier, CA. USA, [13]), International Building

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Code - Edition 2012 (International Code Council, Inc. Falls Church, VA, USA, [14]), National Building Code of Canada. Edition 2010 (National Research Council of Canada, Ottawa, Ont. [15]) and AISI S400 [16].

This work is part of a collaboration between University of Leeds and a UK modular housing company, ilke Homes, that aims to characterise the racking behaviour of industrial CFS modular buildings, and optimise, in particular, the shear walls. The research, overall look at the characterization of the connections between steel profiles and OSB and CP panels, the optimization of shear walls through finite element [17] and experimental studies, and evaluation of racking capacity of walls with openings. Full experimental test matrix is presented in table 1. This paper presents the shear wall tests carried out on CFS walls sheathed only on one side with OSB and CP panels, and having two ledger beams, and no openings. In following sections, the test set up and instrumentation are discussed together with the test result (section 2). A comparison between test results and design is provided (section 3), and finally some recommendation are presented in section 4.

**Table 1** Overall experimental test matrix

| Test typology | N. of tests |
|---|---|
| Tensile steel tests | 32 |
| Shear screw tests | 20 |
| Connection tests | 27 |
| Wall test | 4 |
| Wall tests with opening | 8 |

## 2 Experimental tests

### 2.1 Test setup

Four full scale specimens were tested under horizontal loads in order to characterize experimentally the lateral wall strength and stiffness. In particular, two identical 2.400 m long and 2.974 m long high, able to reproduce in each detail housing ground floor (GF) walls (Fig. 1), and two identical 2.400 m long and 2.926 m long high, able to reproduce in each detail first floor (FF) walls (Fig. 2), were tested.

All the walls are composed of studs, tracks, and blocking profiles made of C100-41-1.6 having 100mm web, 41mm flanges, 10mm lip and 1.6mm thick, with nominal grade 450MPa. Studs are spaced at 600mm. Blockings are placed, in the GF walls, at 610mm from the bottom, mid height of the wall, and at 213mm from the top of the wall. This particular configuration is due to the necessity to use three different sheathing panels, made of 610mm height, 12.5mm thick cement panels (CP) on the bottom part of the wall, and 15mm oriented strand board (OSB3) panels in the other part of the walls. In the FF walls, instead blockings are located at 273 mm from the bottom, at mid height, and at 213mm from the top. 15 mm OSB3 panels are adopted in the FF wall configuration. In all walls, sheathing panels are only placed on one side of the walls. In terms of connections, steel to steel connections are made of 5.5 mm diameter screws, OSB-to-steel and CP-to-steel connections are made of self-drilling screws. Sheathing-to-steel connections spacing varies as follows: in the bottom and top part of the wall, connections are placed every 300mm; in the central part of the wall, connections around the edge of the panels are placed at 150mm spacing, while field screws spacing is 300mm. Shear anchors, made by 16mm diameter bolts placed every 600mm, are used to connect the bottom track to the foundation. Hold-downs are located at each end of the walls to prevent uplift, and for them, Simpson Strong-Tie HTT5 hold-down are used, which are connected to the studs with 26 4.8 mm diameter 16 mm long screws, and to the ground through 16mm diameter bolts. C200-65-1.6 and C150-65-1.6 perimeter beams for, respectively, floor and ceiling cassettes with nominal grade of 450 MPa are located in the inner side of the walls.

The tests were performed in agreement to BS EN 594 (1996) [18], which at the present is adopted in UK for any wall test on both wooden frames and CFS frames. The code prescribes both the specimen arrangement and the loading protocol.

The walls were placed on a composite rectangular hollow beam made by two U sections, 100 x 50 x 10 mm and 150 x 75 x 18 mm, respectively, that were welded to the strong floor. The walls was restrained out of plane, to prevent any out-of-plane displacement.

Three LVDTs were placed to measure the displacements, as shown in Fig. 3. In particular, horizontal displacements on the top and bottom of the wall (LVDT 1, 2) on the side opposite to the applied load were recorded, and vertical displacements at the bottom of the wall (LVDT 3), where the horizontal load is applied were recorded to capture any wall uplift.

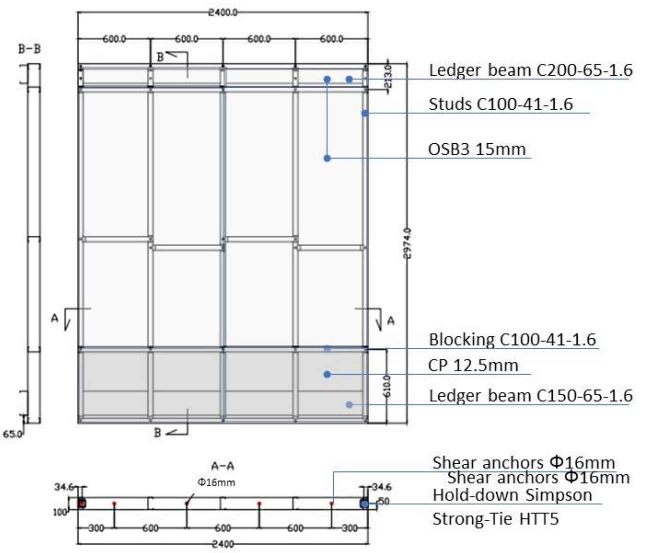


**Figure 1** Ground floor (GF) wall .

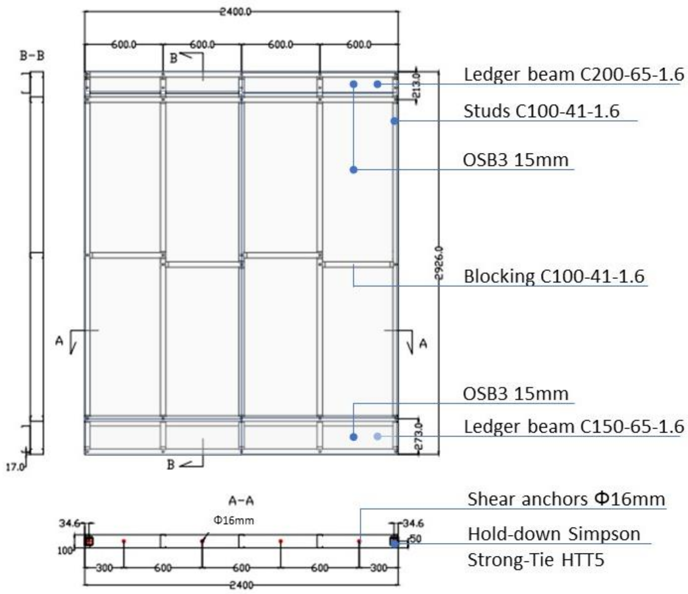


**Figure 2** First floor (FF) wall .

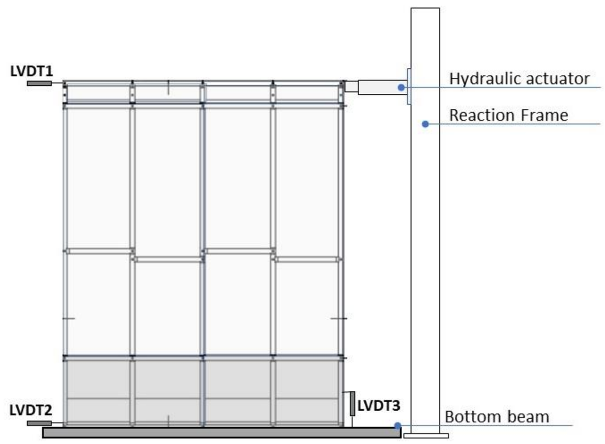


**Figure 3** Wall test set up .

## 2.2 Loading protocol

All tests were displacement-controlled quasi-static loading. Loading in the tests was in accordance to the BS EN 594 (1996) [18]. The required full test cycle is shown in Figure 3. The test method was divided into a stiffness test followed by a strength test. Following a racking preload causing a loading of 10% peak capacity, the stiffness test consists in applying a racking load up to 40% peak capacity of the specimen, after which, the racking load was then removed. This loading cycle was repeated two times and on the second cycle the racking load was increased until failure of the specimen, ensuring that the racking displacement did not exceed 4 mm every 1 minute.

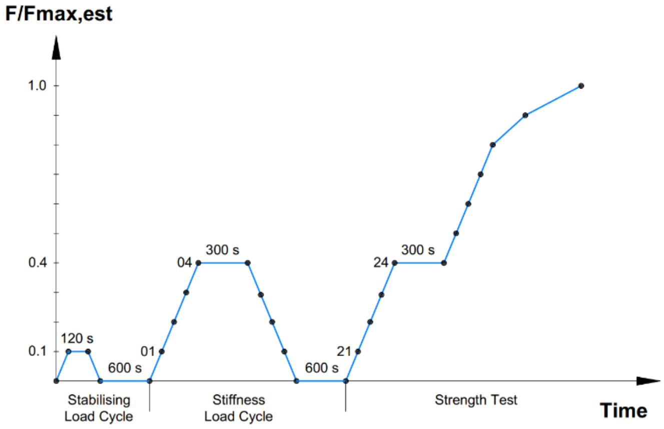


**Figure 4** BS EN 594 (1996) loading protocol for racking monotonic tests.

From the EN 594 (1996) [18] test procedure, racking strength and stiffness is determined. The racking stiffness ($R$, in kN/mm) is determined from the following equation:

$$R = \frac{1}{2} \cdot \left( \frac{(F_{04} - F_{01})}{(v_{04} - v_{01})} + \frac{(F_{24} - F_{21})}{(v_{24} - v_{21})} \right) \quad (1)$$

Where $F_i$ is the racking load from the test load cycle (in kN) and $v_i$ is the racking displacement under $F_i$ (in mm). As for the racking strength, it is represented by the peak capacity of the tested assembly.

## 2.3 Test results

Test results revealed that for all GF specimens, the wall collapse was governed by the sheathing-to-frame connections as shown in Figure 7. At global level, the steel frame deformed as a parallelogram with a consequent rigid rotation of the sheathing panels, that determined first the titling and pull through of all the screws, that was followed by the cracking of the CP, and the edge breaking of the CP panels corners.

The FF panels exhibited a similar failure mechanism (Fig. 8), with the steel frame deforming into a parallelogram, the rigid rotation of the panel, and the tilting and pull through of the connections. However, the final failure was due to buckling of the stud, in correspondence of the applied load. Clearly this failure was induced by the application of the horizontal load on the web of the first stud.

The result of each test is presented in Figure 5 and 6 in terms of racking strength versus top displacement. Table 2 also summarises the results, and indicates that the GF walls exhibited an average maximum strength equal to 41.79kN and average stiffness of 2.32 kN/mm, while the FF walls showed an average strength equal to 62.48 and stiffness equal to 2.04kN/mm. Therefore the walls with only OSB panels have a shear strength at least 1.5 time higher than a similar walls with CP panels in the bottom part.

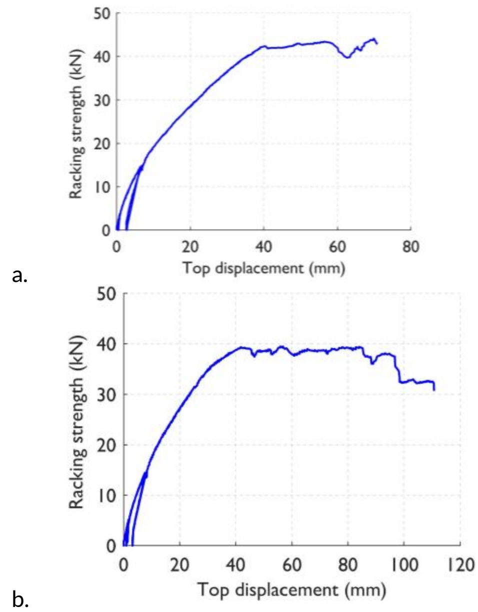


**Figure 5** Strength to displacement curve for: a) Test n.2, b) test n.3 of GF wall.

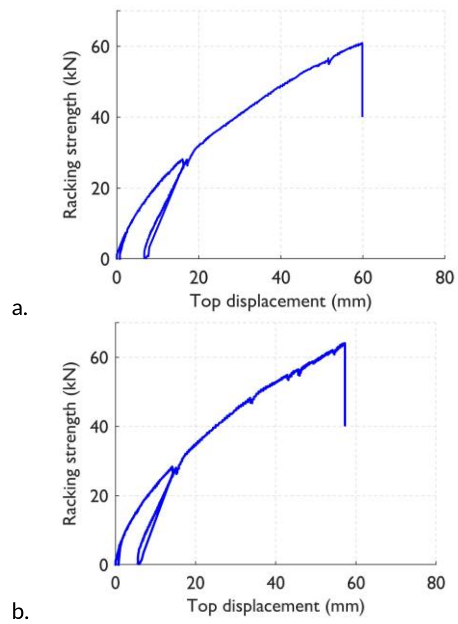


**Figure 6** Strength to displacement curve for: a) Test n.2, b) test n.3 of FF wall.

Table 2 Evaluation of strength and stiffness of the tested walls

| Test Label | Width [m] | Height [m] | Experimental $F_{peak}$ [kN] | Displacement $d_{peak}$ [mm] | Stiffness [kN/mm] |
|---|---|---|---|---|---|
| GF 2 | 2.4 | 2.974 | 44.12 | 50.73 | 2.48 |
| GF 3 | | | 39.46 | 55.96 | 2.17 |
| average | | | 41.79 | | 2.32 |
| FF 2 | 2.4 | 2.926 | 60.92 | 59.86 | 1.95 |
| FF 3 | | | 64.04 | 57.24 | 2.13 |
| average | | | 62.48 | | 2.04 |

**Figure 7.** Failure mode of GF walls

**Figure 8.** Failure mode of FF walls

## 3 Comparison with design estimation

Shear strength of the tested wall panels was estimated at design stage according to the SCI ED002 [19], which provides rules for the evaluation of wall design resistance. The SCI ED0002 evaluation is based on the assumption that: a. the connections between studs and sheathing have a linear behaviour, b. the wall racking resistance is directly proportional to the spacing of the fasteners, c. there is no moment transferred between steel members, d. any lifting is prevented. Based on these assumption, it defines the shear resistance, as provided in equation 2:

$$F_{RD} = \frac{f_{rv}B}{\gamma_M \gamma c} \tag{2}$$

Where $f_{rv}$ is the connection strength experimentally determined, B is the wall width, $\gamma_M$ is the material safety factor, and it is considered equal to 1.3, c is the fastener spacing, and γ is determined according to table 4.1 of SCI ED0002 [19], and for the specific wall under investigation is given by equation 3:

$$\gamma = \sqrt{\frac{36}{(6+5\frac{H}{B})^2} + \frac{144}{(4\frac{B}{H}+15)^2}} \tag{3}$$

Where *H* is the height of the wall, and *B* is the width of the wall.

Following this methodology, the estimated design shear resistance, considering the fastener spacing (c) equal to 150mm, and $f_{rv}$ equal to 2.11 kN, as evaluated from experiments described in [20] is 31.60kN for GF walls, and 31.56 kN for FF walls.

When compared to the racking strength recorded in the experiments, the experimental values are respectively 1.32 and 1.98 times higher than the design values. This shows that the current UK code is very conservative in the evaluation of design strength of the sheathed braced CFS walls.

## 4 Conclusions and future work

Advancement of CFS in industry requires advancement of design codes. At the present, Europe lacks of a specific code that allows the design of CFS walls under horizontal loads. This becomes, even more critical, when the collaboration between CFS members and sheathing panels want to be considered. This approached is known as "Sheathing braced design" methodology.

The research presented in this paper aims to advance the understanding of racking capacity of CFS walls sheathed on one side with OSB and CP panels. Two walls configurations are investigated, having width 2.4m and height equal to 2.974 and 2.926m, respectively. Both wall typology also have two ledger beams. The walls are tested according to the BS EN 594 [18], and the results shows that, in both cases the failure mechanism is governed by CFS- to- sheathing connection failure. In particular, in the first wall typology (GF), the failure is due to tilting and pull through if the screws, followed by the breaking of CP panels edges. This is due to the fragile response of CP panels. As for the second typology of walls (FF), while the failure mechanism is initiated by the tilting and pull through the OSB of the connections, a sudden failure is reached with the loading jack instigating the buckling of the stud, where the load is applied. This mechanism is clearly due to the loading set up. Indeed, while in previous research [4, 7-9] the in plane load is applied to CFS walls through a spreading beam located on the top of the CFS walls, in the presented research, since the BS EN 594 has been followed, no spreading beam has been included. The results demonstrate that, despite the BS EN 594 is required in UK to be followed for the testing of CFS walls, appropriate improvement of the test set up is required to be able to capture the full shear capacity of CFS walls sheathed with OSB. The GF wall tests, also show, that even when the failure mechanism is reached, the wall has still a residual shear capacity (see plateau in the curve). The comparison between shear capacity between the two investigated wall typologies, also shows that GF typology, which in particular include CP panels, and bottom strip with height 617mm, had a much lower racking capacity. The FF walls, indeed, fully sheathed with OSB3 panels, and with the bottom strip of 273mm height, has a shear capacity which is at least 1.5 times higher.

Finally the experimental results have been compared to the design shear resistance, evaluated in agreement with the SCI ED002. The comparison shows that the obtained design values are strongly conservative. This outcome reinstates the necessity to develop ad-hoc codes for the design of CFS structures.

## 5 Acknowledgments

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