Experiments on sheathed cold-formed steel beam-columns

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Abstract
An experimental study on sheathed cold-formed steel C-lipped columns (studs) with service holes in the web subjected to compression and major axis bending is presented in this paper. A total of 17 experiments was performed with both oriented strand board (OSB) and plasterboard used as the sheathing material and with varying connector spacing employed between the sheathing and the steel members. Material tests and geometric imperfection measurements were also undertaken. The tested specimens comprised a single 2.4 m long cold-formed steel member sheathed on both sides and secured at its ends to top and bottom tracks. Eight pure compression tests with plasterboard and OSB sheathing and with the spacing of the connectors varying between 75 mm and 600 mm were initially performed. Specimens with OSB were then tested under pure bending and also under combined loading, by first applying a compressive load corresponding to the 75%, 50% or 25% of the predetermined axial resistance and then laterally loaded until failure. Reducing the spacing of the connectors from 600 mm to 75 mm resulted in up to 20% and 30% increases in capacity for the studs sheathed with OSB and plasterboard respectively.

Keywords
Beam-columns, Cold-formed steel, Experiments, Plasterboard, Stud columns, OSB

1 Introduction

Wall systems made out of cold-formed steel C-lipped vertical members (studs) set in tracks and sheathed with wood-based boards or gypsum plasterboards are becoming increasingly common in the construction industry. The versatility, practicality and economy associated with the quick construction process of these systems allows for their use both as non-structural and load-bearing wall systems. When used as load bearing systems, additional cold-formed steel members are provided as lateral bracing, as shown in Figure 1 (a), although several studies [1, 2] have shown that the attached sheathing (Figure 1 (b)) can provide sufficient bracing, resulting in higher axial and flexural capacities. The focus of this study is to extend the existing research and demonstrate that using closer spaced connectors between the column and the sheathing can further enhance the capacity of these systems under both compression and bending and can prevent the sudden and brittle failure of sheathing-to-steel connectors.

Increased compression capacities of cold-formed steel columns, achieved due to their connection with sheathing, have been experimentally demonstrated in various studies [3–7]. Studies on gypsum sheathed stud-members subjected to major axis bending [8] have highlighted that, if pull-through fastener failures are prevented, significant enhancements in terms of load-bearing capacity can be achieved. Improvements in the resistance of wall-stud assemblies subjected to lateral loading, both compression and major axis bending when OSB and gypsum sheathing is added have also been experimentally demonstrated in [9].

The enhancements in load-bearing capacity, resulting when decreasing the connector spacing between sheathing and steel [5, 8, 9] are believed to arise due to greater restraining effects and mobilisation of composite action between the two materials. Experimental work on load-bearing cold-formed steel stud columns braced only by sheathing, and subjected to compression and major axis bending has been performed in [9]. Single-sided OSB sheathed studs and doubly-sided sheathed studs with OSB and gypsum boards under various combinations of compression load and bending until failure have been assessed against results from bare steel studs.

In the experimental study presented herein, sheathed stud columns, representative of a single vertical element in a stud wall system [1, 2], are tested in both compression and bending – see Figure 1 (c). Note that the tested members also feature service holes, as shown in Figure 2. Material and geometric characteristics are provided in Section 2, while imperfection measurements along with details of the components of the specimens and of the test rig are reported in Section 3. Results are presented in Section 4 with a discussion following in Section 5.

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2 Material tests

The stress-strain behaviour and key materials properties of the main components of the examined sheathed stud columns, as shown in Figure 1(c), were determined by material coupon testing.

2.1 Cold-formed steel

For the determination of the material properties of the stud and track, three coupons were extracted along the length of the web of both members. The notation of the coupons begins with the letters CS, followed by the nominal thickness in mm multiplied by 10 (CS15 for the studs and CS12 for the track) and then the test number. The tensile coupons were tested in accordance with EN ISO 6892 [10] at an initial axial strain rate of 0.00007 s⁻¹ which was gradually increased to 0.00025 s⁻¹ via three intermediate steps. A 150 kN Instron 5984 testing machine was used to perform the tests, along with the Bluehill data acquisition system, collecting data at every second. Accurate measurement of strains at the early loading stage was ensured with two strain gauges, mounted at mid-height on either side of the coupon, while an optical extensometer with a gauge length of 50 mm was used as an additional strain measurement method to capture the later parts of the stress-strain curves.

The key measured properties are presented in Table 1 while the full stress-strain curves are illustrated in Figures 3 and 4 for the stud and track respectively. In Table 1, \( E \) is the elastic modulus, \( \sigma_{0.2} \) is the 0.2% proof stress (considered as the yield strength), \( \sigma_{0.2} \) is the 1% proof stress, \( \sigma_u \) is the ultimate strength, \( \varepsilon_0 \) is the strain at \( \sigma_0 \) and \( \varepsilon_f \) is the elongation at fracture measured over the standard gauge length [10]. It can be observed that while the yield strengths of the stud and track have similar values, the values of \( \varepsilon_0 \) and \( \varepsilon_f \) of the track are 20% and 60% higher than those of the stud, respectively.

### Table 1 Results of cold-formed steel tensile coupons on material extracted from columns (CS15) and tracks (CS12)

| Coupon ID | \( E \) (GPa) | \( \sigma_{0.2} \) (MPa) | \( \sigma_{u} \) (MPa) | \( \varepsilon_0 \) (%) | \( \varepsilon_f \) (%) |
|-----------|---------------|------------------------|----------------------|----------------------|----------------------|
| CS15-1    | 203           | 481                    | 514                  | 0.693                | 0.121                |
| CS15-2    | 203           | 484                    | 514                  | 0.892                | 0.148                |
| CS15-3    | 205           | 481                    | 514                  | 0.785                | 0.120                |
| Average   | 204           | 482                    | 514                  | 0.790                | 0.130                |
| CS12-1    | 201           | 483                    | 613                  | 0.137                | 0.222                |
| CS12-2    | 201           | 483                    | 609                  | 0.124                | 0.195                |
| CS12-3    | 201           | 484                    | 618                  | 0.140                | 0.229                |
| Average   | 201           | 483                    | 613                  | 0.134                | 0.215                |

### Table 2 Average measured material properties of sheathing

|| Longitudinal | Transverse |
|--------------|------------|------------|
| Type         | \( E \) (GPa) | \( f_s \) (MPa) | \( \varepsilon_0 \) (%) | \( \varepsilon_{0.2} \) (%) | \( E \) (GPa) | \( f_s \) (MPa) | \( \varepsilon_0 \) (%) | \( \varepsilon_{0.2} \) (%) |
| OSB, tension | 4.10       | 11.7       | 0.37       | -                     | 2.94          | 7.01        | 0.37        | -                     |
| OSB, compression | 4.02     | 12.1       | 0.40       | 0.54                  | 2.78          | 10.7        | 0.50        | 0.66                  |
| Plasticboard, tension | 1.65     | 1.45       | 0.61       | -                     | 1.67          | 0.67        | 0.38        | -                     |
| Plasticboard, compression | 2.11      | 3.81       | 0.23       | 0.37                  | 2.13          | 3.33        | 0.23        | 0.42                  |
Table 3 Average measured cross-sectional dimensions

| Member | t (mm) | h (mm) | b (mm) | c (mm) | r (mm) |
|--------|--------|--------|--------|--------|--------|
| Column | 1.44   | 149.15 | 43.92  | 11.95  | 2.50   |
| Track  | 1.18   | 155.30 | 31.18  | -      | 2.25   |

### 3.2 Imperfections

Initial geometric imperfections, including both cross sectional ($\delta_l$ for local, $\delta_D$ for distortional) and global ($\delta_{G, maj}$, $\delta_{G, min}$, $\theta_{G}$) modes along the length of the member, were measured for ten specimens with service holes (as shown in Figure 2) and ten without service holes. The five modes of imperfection, illustrated in Figure 6, were determined using an imperfection rig with seven LVDTs – see Figure 7 – which allowed the measurement of the variation of the shape of the cross section along the length of the 2.4 m long specimens. The method used to determine the imperfection magnitudes was similar to the one described in [6]. The seven LVDT mount ran on rails, as shown in Figure 8, with an attached string pot allowing determination of the position of the measurements along the length of the column. The rails on the imperfection rig (Figure 8) were calibrated to remove their inherent imperfection. Data were recorded using the acquisition system DATASCAN [14] every second.

The imperfections along the length of the columns were calculated using the following equations, where $\delta_L$, $\delta_D$, $\delta_{G, maj}$, $\delta_{G, min}$ and $\theta_G$ are as defined in Figure 6 and $\delta_4$, $\delta_7$ are the measurements of the LVDTs shown in Figure 7.

\[
\delta_L = \text{mean}(\delta_4 - \delta_3, \delta_6 - \delta_5) \quad (1)
\]

\[
\delta_D = \text{mean}(\delta_1 - \delta_2, \delta_7 - \delta_5) \quad (2)
\]

\[
\delta_{G, maj} = \text{mean}(\delta_2, \delta_6) \quad (3)
\]

\[
\delta_{G, min} = \text{mean}(\delta_5, \delta_7) \quad (4)
\]

\[
\theta_G = (\delta_3 - \delta_5)/h \quad (5)
\]

The average imperfection amplitudes of the specimens with and without service holes are provided in Table 4, as a multiple of the thickness for the cross sectional modes and as a multiple of the column length L for the global modes. Typical distributions of the imperfections along the member length are presented in Figures 9 and 10 for the cross-sectional and global imperfections respectively. In Figure 9, all data have been post-processed so that the measurements are plotted relative to a best fit line through the overall imperfection profile. The imperfection amplitude for the cross-section modes is then taken as the maximum deviation from this line. For the global imperfections, a half-sine wave was fitted to the measured curves using least squares regression to allow determination of the maximum amplitude at mid-length, as shown in Figure 10.
The imperfection measurements for the members with and without holes show no significant discrepancies for the local, distortional and twist modes. The global flexural imperfections along the major axis for both members are of the low order of $L/10000$, indicating no significant cambering. The magnitude and direction of bow of the global flexural imperfections along the minor axis is notably different between the members with and without holes; this is attributed to hole punching during manufacturing. Note that, for columns with holes, the positive imperfection value denotes that the deflection is as per the illustration in Figure 6 (c), while for columns without holes the negative value indicates the imperfection is in the opposite direction.

3.3 Screw types and arrangements

The types of connector used for securing the sheathing panels to the steel studs were screws of 3.5 mm diameter for the plasterboard and 4.8 mm diameter for the OSB. Illustrations of the two screws are provided in Figure 11 (a) and (b) respectively, while further details on the properties of these screws and on their behaviour in shear and pull-through can be found in [11]. The connection between the flanges of the track and stud at both column ends were implemented using screws of 4.8 mm diameter - see Figure 11 (c). Finally the webs
of both tracks were secured onto square hollow sections, using bolts of 6 mm diameter (see Figure 11 (d)).

![Figure 11 Types of connector used: (a) 3.5 mm diameter screw for plasterboard to steel (b) 4.8 mm diameter screw for OSB-to-steel (c) 4.8 mm diameter screw for steel-to-steel and (d) 6 mm diameter anchor bolt.](image)

The sheathed column specimens were tested with four different screw spacings of 600 mm, 300 mm, 150 mm and 75 mm, as shown in Figure 12. Note that a distance of 50 mm was kept between the edge connector and the column end, in order for the screw to not be driven into the track. For the spacing of 600 mm, five screws were used on each side of the column, with the two outer screw spacings reduced to 550 mm. Similarly, for the other arrangements, the outer spacings were reduced to allow for a uniform spacing of connectors along the rest of the column length. For the 300 mm spacing, nine screws were used, while for the 150 mm and the 75 mm spacings, 17 and 33 screws were employed respectively. No connection between the track and the sheathing was implemented since the focus of this study was to investigate the connection between column and sheathing.

![Figure 12 Connector arrangements for spacings of (a) 75 mm, (b) 150 mm, (c) 300 mm and (d) 600mm (dimensions in mm).](image)

### 3.4 Test rig

The specimens were tested in a dual-actuator rig. Axial compression was applied to the overhead track by a vertical 1000 kN Instron actuator while bending was applied by laterally loading the sheathing using a four-line two tier whiffletree system connected to an horizontal 500 kN Instron actuator in order to approximate a uniformly distributed load - see Figure 13. The top actuator was connected to a hot-rolled square hollow section to ensure an even load distribution at the top track of the tested specimen during loading. Note that special attention was given to avoid contact between the square hollow section and the sheathing (such that axial load would be applied on the steel stud only). Rotation of the actuator and of the hollow section was prevented with the use of brackets at either end of the hollow section, while Teflon was placed between the brackets and the hollow section to minimise friction. The bottom track was also secured to a hollow section fixed to the concrete lab floor. The whiffletree connected to the horizontal actuator was free to rotate, with pinned connections between its legs. Prior to each bending test, the whiffletree was laser-aligned to be parallel to the column length.

![Figure 13 Schematic view of test rig](image)

In total 19 LVDTs and one string potentiometer were used to measure displacements. Eight LVDTs, along with the string-pot attached to the column, were used to measure the shortening of the column. At each column end, two LVDTs were used to measure the deflection of the track, while two LVDTs mounted on either side of the column measured the closing gap between the column and the track during loading. The net axial shortening of the column was calculated as the difference between the gap closure and track deflection. A second method that was also used to measure the axial shortening...
of the column was the string potentiometer spanning between the ends of the column, as shown in Figure 14. Ten LVDTs on the laterally loaded side and one in the centre of the non-loaded side were used to measure lateral displacements. The LVDTs on the loaded side were positioned in the middle of the sheathing and as close as possible to the loading arms of the whiffletree to record the maximum displacement due to bending, as shown in Figure 14. The loads and displacement measurements from the actuators, the LVDTs and the string potentiometer were captured using the DATASCAN [14] acquisition system, recording at a frequency of 1 Hz.

3.5 Boundary and loading conditions

The length of the tracks of the tested specimens was 800 mm, as shown in Figure 14 (b). Detailed views of the top and bottom tracks connected to the test rig are shown in Figure 15, where also the 6 mm anchor bolts with washers are shown; these were used to secure the tracks to the hot-rolled sections. The connection between the column and the track is shown in Figure 16.

Although at the beginning of the tests the assumption of pinned boundary conditions was deemed to be valid, it was found that, during loading, the increasing axial force results in direct contact between the column end and the web of the track. Further increases in axial force can lead to restraint of all translations and rotations of the end cross-sections (i.e. at the top and bottom of the stud) resulting in the boundary conditions being closer to a fixed (rather than pinned) configuration.

The four-point whiffletree system was constructed so that its loading arms would have a uniform contact area across the width of the sheathing board to resemble four line loads, which approximate a uniformly distributed load along the length of the column. The bending moment diagram under the four point loads and under an equivalent uniformly distributed load assuming pinned end conditions is shown in Figure 17 (a), while, in Figure 17 (b), the corresponding bending moment diagrams assuming fixed boundaries are shown.

![Figure 16 Column-to-track connection](image1)

![Figure 17: Bending moment diagrams under 4-point loads and an equivalent uniform distributed load for (a) pinned and (b) fixed boundary conditions](image2)

The axial load was applied using displacement control at a rate of 0.25 mm/min while the horizontal load was applied at a rate of 2.00 mm/min. The specimens subjected to pure compression or pure bending were loaded beyond the ultimate load, until a load decrease could be clearly observed. For the combined load cases, the specimens were first axially loaded to a pre-determined load level which was kept constant and then horizontal load was applied until failure occurred. During horizontal loading, shortening of the column meant than, in order to keep the axial load constant, the load had to be regularly checked and manually adjusted (by increasing the axial displacement of the top actuator).

4 Experimental results

A total of 17 tests were conducted, with eight specimens subjected to pure compression, six to combined compression and bending and three to pure bending. The tested stud columns were sheathed on both sides with the same type of board and the same number of screws on each side. A summary of the tests, along with details of the type of sheathing used, the spacing between screws and the axial load $P_a$ or horizontal load $H_a$ at failure, is provided in Table 5. The notation used for each test in Table 5 begins with the letter S for plasterboard (gypsum) or O for OSB, then with the level of the applied compressive load (expressed as a percentage of the compressive strength of the stud), followed by the letter S and finally by the spacing between screws in mm. For specimens which were repeated, 1 or 2 was added at the end of their notation to differentiate between them.

Six of the specimens under pure compression were sheathed with plasterboard and two with OSB. For the specimens with plasterboard, all four connector spacings were examined, as illustrated in Figure 12, while, for the specimens with OSB, only the two extreme
spacings of 600 mm and 75 mm were studied. One repeat test was conducted for the specimens with plasterboard for the 600 mm and 300 mm connector spacings.

Following testing of the plasterboard specimens under pure compression, and since pull-through failure was prominent, it was decided that the specimens to be tested under combined loading and pure bending would be sheathed with OSB only, with connectors spaced either at every 600 mm or 75 mm. Six tests were conducted under combined load, where the axial compression was set to 75%, 50% and 35% of the stud compression resistance, which had been determined based on the two tests with OSB sheathing under pure compression. Three tests were conducted under pure bending, since the 75 mm connector spacing case was repeated.

Table 5 Summary of details of the tested specimens and observed failure loads

| #  | Name          | Sheathing | Screw spacing (mm) | $P_u$ (%) | $H_u$ kN |
|----|---------------|-----------|--------------------|-----------|----------|
| 1  | G-100-S600-1  | Gypsum    | 600                | 100       | 69.0     | 0.0      |
| 2  | G-100-S600-2  | Gypsum    | 600                | 100       | 77.7     | 0.0      |
| 3  | G-100-S300-1  | Gypsum    | 300                | 100       | 76.5     | 0.0      |
| 4  | G-100-S300-2  | Gypsum    | 300                | 100       | 82.1     | 0.0      |
| 5  | G-100-S150    | Gypsum    | 150                | 100       | 85.5     | 0.0      |
| 6  | G-100-S75     | Gypsum    | 75                 | 100       | 90.5     | 0.0      |
| 7  | O-100-S600    | OSB       | 600                | 100       | 79.5     | 0.0      |
| 8  | O-100-S75     | OSB       | 75                 | 100       | 90.7     | 0.0      |
| 9  | O-75-S600     | OSB       | 600                | 75        | 60.0     | 13.9     |
| 10 | O-75-S75      | OSB       | 75                 | 75        | 67.5     | 15.5     |
| 11 | O-50-S600     | OSB       | 600                | 50        | 40.0     | 17.4     |
| 12 | O-50-S75      | OSB       | 75                 | 50        | 45.0     | 21.3     |
| 13 | O-25-S600     | OSB       | 600                | 25        | 20.0     | 20.8     |
| 14 | O-25-S75      | OSB       | 75                 | 25        | 22.5     | 23.5     |
| 15 | O-0-S600      | OSB       | 600                | 0         | 0.0      | 20.9     |
| 16 | O-0-S75-1     | OSB       | 75                 | 0         | 0.0      | 22.4     |
| 17 | O-0-S75-2     | OSB       | 75                 | 0         | 0.0      | 20.8     |

### 4.1 Pure compression

A summary of the results of the pure compression tests is shown in Figure 18, where the failure load $P_u$ is plotted against the employed spacing of connectors. Substantial increases in capacity can be observed with decreasing the spacing of the connectors (from 600 mm to 75 mm), with a 14% increase attained for the OSB sheathed specimens and an average 23% increase for the plasterboard-sheathed specimens. Note that an average trend line has been plotted for the plasterboard specimens due to the variation between the results of the repeated tests with screw connectors at 600 mm and 300 mm. For all of these specimens, pull-through failure of the connectors was observed, triggered by distortional buckling of the flanges pushing the boards away (see Figure 19 (a)) and then followed by sudden flexural torsional buckling of the column once the connection to the sheathing was lost. For the specimen sheathed with plasterboard and connected at 150 mm intervals, distortional buckling near the top end of the column led to the top end screws failing by pull-through; this then led to further development of local and distortional buckling and eventually to failure. Finally, for the specimens with a connector spacing of 75 mm, pull-through of the screws was prevented and failure was triggered by local buckling near the bottom end of the column. Regarding the specimens sheathed with OSB, these both suffered a combination of distortional and local buckling, with the specimen sheathed with OSB at every 75 mm reaching a higher capacity due to the higher restraint provided by the screw connectors.

The load-displacement curves of a typical specimen under compression (G-100-S75) are shown in Figure 20, where each different curve corresponds to a different method used for the determination of the axial displacement (i.e., measurements from the vertical actuator, the string potentiometer attached to the column and the difference between the measurements of the LVDTs positioned on the tracks and on the column measuring the gap between them). As shown in Figure 20, the displacement recorded by the actuator compares poorly with the theoretical axial stiffness of the column $EA/L$ (where $E$ is the elastic modulus of the steel, $A$ the cross sectional area of the column and $L$ the length of the column), with the gradual increase of stiffness at the beginning of the curve corresponding to the gradual closure of the gap between the track and the column. On the other hand, since the string potentiometer can accurately capture the initial shortening of the column during loading, the curve derived based on its measurements initially matches the theoretical axial stiffness. Deviations at higher load relate to the asymmetric nature of the measurements taken on one side of the specimens only. Finally, the curve calculated based on the difference of the LVDT readings measuring the deflection of the tracks and the gap closure between the track and column (see Figure 14 (b)) matches closely the actual shortening of the column. Based on these observations, it was concluded that, for all specimens, the three different types of measurements had to be combined in order to obtain the most representative load-end shortening curves; these are presented in Figures 21-23.
In Figure 21, the load-displacement curves of the OSB sheathed specimens are presented while in Figure 22, the curves of the specimens sheathed with plasterboard are provided. It can be observed that the curves corresponding to all of the specimens comprising OSB sheathing and to the specimen sheathed with plasterboard with the connectors spaced at 75 mm are smooth; this is because connector failure did not occur prior to the attainment of the peak load of the system. Conversely, the curves of the specimens which suffered premature pull-through failure of the connectors exhibited small steps (jumps) in the response.

A comparison between the specimens sheathed with OSB and plasterboard is presented in Figure 23. As expected, it can be observed that the stiffness and capacity of the specimens increases with increasing number of connectors due to the additional restraint to both the global and cross-sectional buckling modes. The specimens with the connectors spaced at 75 mm intervals exhibited similar peak capacities, although a slightly higher stiffness was observed for the specimens sheathed with OSB.

The failure modes of the columns sheathed with plasterboard are shown in Figure 24. For the specimens that were connected at 600 mm and 300 mm intervals, distortional buckling developed between the connectors (see Figure 24) while the specimens sheathed at every 150 mm and 75 mm suffered buckling near the column ends.

4.2 Compression and bending

In Figure 25, the ultimate compression load $P_u$ is plotted against the ultimate horizontal load $H_u$ for the specimens sheathed with OSB (and with connectors spaced at 600 mm and 75 mm intervals). Increased capacities in both compression and bending (between 15 – 20%) were attained when closer-spaced connectors were employed. It should be mentioned that the capacity of the specimens tested under pure bending was limited by the strength of the connection between the track and the column since, due to severe deflection at the bottom end of the column, the screw sheared off for the specimens tested with screws at 75 mm spacings.

In Figures 26 and 27, the horizontal load is plotted against the horizontal mid-height displacement for all tested specimens. Note that the horizontal mid-height displacement was calculated by averaging the three LVDT measurements at mid-height (from the two LVDTs
on the loaded side and the third one on the non-loaded side – see Figure 14(b). The full load-deflection curves are shown in Figure 26 while their initial part is presented in Figure 27, where the lines corresponding to the theoretical pinned and fixed boundary conditions are also plotted. It was found that the stiffness of the specimens with connectors spaced at 75 mm intervals that were subjected to an axial load corresponding to 75% and 50% of their compressive resistance was approximately 20% higher than the theoretical stiffness corresponding to an equivalent column with pinned-end conditions. Similarly, the response of the specimens with connectors spaced at 600 mm intervals under an axial load equal to 75% and 50% of their compressive resistance was 10% stiffer compared to the theoretical response of a column pinned at both ends. The remainder of the results matched the pinned theoretical stiffness.

The OSB-sheathed specimens that were subjected to combined loading exhibited local failure at the service openings and at the ends of the column as shown in Figure 28. This can be explained with reference to the location of the maximum bending moment for a pinned-ended member under four-point loads. In accordance with Figure 17, this critical location spreads over the central quarter length, where both service holes are located. For the specimens sheathed with OSB at 600 mm spacings and subjected to pure bending, failure did not occur near the service holes but at the location where the bottom line load of the whiffletree was applied. Finally, both specimens with connectors spaced at 75 mm under pure bending exhibited screw failure at the end of the column – see Figure 28 (b).

![Figure 25](image_url) Ultimate compression ($P_c$) and horizontal ($H_e$) loads for specimens sheathed with OSB and connected at 600 mm and 75 mm intervals

![Figure 26](image_url) Horizontal load versus mid-height displacement curves

![Figure 27](image_url) Initial part of the horizontal load versus the mid-height displacement curves and comparison with the theoretical fixed and pinned boundary conditions

![Figure 28](image_url) Failure modes of all the tests specimens (from left to right – 100% compression to 100% bending) with OSB sheathing connected at (a) 600 mm spacings and (b) 75 mm spacings

5 Discussion

The compression capacities of the sheathed specimens, even for the widest connector spacing, were found to be significantly higher than those predicted using the currently available design methods [15]. The typical assumption of pinned boundary conditions for studs under pure compression, in combination with the assumption of no restraint offered by the presence of the sheathing, results in an underestimation of the member capacities, with the level of conservatism depending on the type of boarding, the member dimensions and the fixing arrangements.

The work by Vieira et al. in [1], in which the connectors were found to provide restraint to global buckling modes, has been verified by the findings of this research while the upper-bound capacity estimations (assuming full restraint to global modes, thus limiting capacity only due to local/distortional buckling) are in agreement with the results of the specimens with the more substantial connection. In addition, as also indicated by Peterman et al. [2, 9], it was found that for studs under combined loading, the assumption of pinned end conditions is valid for axial loads up to 50% of the compression resistance with a slight increase towards fixity for axial loads of about 75% of the compression resistance.

Numerical analysis to support the findings of this study is underway.
aiming at the establishment of a design methodology explicitly recognising the contribution of sheathing.

6 Conclusions

An experimental study of sheathed cold-formed steel members is presented in this paper. Seventeen full-scale tests comprising eight specimens under pure compression, six specimens under combined loading and three under pure bending were carried out, along with material tests and imperfection measurements. The tested specimens were sheathed with either plasterboard or OSB, on both sides, with various stud-to-sheathing connector spacings ranging from 600 mm to 75 mm.

When subjected to pure compression, the failure mode exhibited by the studs connected to plasterboards at the wider spacings (i.e. 600 mm and 300 mm) was pull-through failure of the connectors, triggered by distortional buckling of the column flanges, then followed by sudden flexural torsional buckling (once the connection to the sheathing was lost). Under pure compression, the observed failure mode for the studs sheathed with OSB and plasterboard at denser connector spacings (i.e. 150 mm and 75 mm) was local buckling at the column ends. Finally, the OSB-sheathed specimens that were subjected to combined loading exhibited local failure at the service openings while, when under pure bending, local buckling and stud-to-track connector failure occurred.

Reducing the spacing of the connectors from 600 mm to 75 mm resulted in up to 20% increases in the capacity of the specimens sheathed with OSB under both compression and bending. For the specimens sheathed with plasterboards and subjected to pure compression, reducing the connector spacing from 600 mm to 75 mm resulted in up to 30% increases in capacity while pull-through connector failure was also prevented.

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References

[1] Vieira Jr, L. C. M.; Schafer, B. W. (2013) Behavior and design of sheathed cold-formed steel stud walls under compression. Journal of Structural Engineering, 139 (5), 772–786.

[2] Peterman, K. D.; Schafer, B. W. (2014) Sheathed cold-formed steel studs under axial and lateral load. Journal of Structural Engineering, 140 (10), 04014074-1–04014074-12.

[3] Simaan, A.; Pekoz, T. (1976) Diaphragm braced members and design of wall studs. Journal of the Structural Division, ASCE, 102 (1), 77–92.

[4] Miller, T. H.; Pekoz, T. (1994) Behavior of gypsum-sheathed cold-formed steel wall studs. Journal of Structural Engineering, 120 (5), 1644–1650.

[5] Telue, Y.; Mahendran, M. (2004) Behaviour and design of cold-formed steel wall frames lined with plasterboard on both sides. Engineering Structures, 26 (5), 567–579.

[6] Vieira Jr, L. C. M.; Shifferaw, Y.; Schafer, B. W. (2011) Experiments on sheathed cold-formed steel studs in compression. Journal of Constructional Steel Research, 67, 1554-1566.

[7] Fratamico, D. C.; Torabian, S.; Zhao, X.; Rasmussen, K. J. R.; Schafer, B. W. (2018) Experimental study on the composite action in sheathed and bare built-up cold-formed steel columns. Thin-Walled Structures, 127, 290-305.

[8] Selvaraj, S.; Madhavan, M. (2018) Studies on cold-formed steel stud panels with gypsum sheathing subjected to out-of-plane bending. Journal of Structural Engineering, 144 (9), 04018136-1–04018136-18.

[9] Peterman, K. D. (2012) Experiments on the stability of sheathed cold-formed steel studs under axial load and bending. MSc thesis Johns Hopkins University Baltimore, Maryland, USA.

[10] EN ISO 6892 (2016) Metallic materials - Tensile testing. Part 1: Method of test at room temperature. European Committee for Standardization. Brussels, Belgium.

[11] Kyprianou, C.; Kyvelou, P.; Gardner L.; Nethercot, D. A. (2020) Characterisation of material and connection behaviour in sheathed cold-formed steel wall systems - Part 1: experimentation and data compilation. Structures. [In progress]

[12] EN 520 (2009) Gypsum plasterboards - Definitions, requirements and test methods. European Committee for Standardization. Brussels, Belgium.

[13] EN 300 (2006) Oriented Strand Boards (OSB) Definitions, classification and specifications. European Committee for Standardization. Brussels, Belgium.

[14] Datascan (computer software). Datascan. Carrollton. Texas. USA.

[15] EN 1993-1-3 (2006) Eurocode 3 – Design of steel structures – Part 1-3: General rules – Supplementary rules for cold-formed members and sheeting. European Committee for Standardization. Brussels, Belgium.