Experimental & Numerical Investigation of an Innovative, High Capacity Cold-Formed Steel Shear Wall

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Abstract

The study presented herein is concerned with establishing benchmark finite element models of high capacity cold-formed steel (CFS) shear walls. CFS shear walls have emerged as an economic and light-weight seismic force resisting system (SFRS), unfortunately their applications are limited to low- and mid-rise residential and commercial buildings. To advance the state-of-the-art, a preliminary, full-scale testing program of an innovative, higher-capacity CFS shear wall is conducted. The shear wall configuration consists of a thin steel sheathing concentrically confined between built-up hat section wall studs and built-up, L-shaped tracks. Furthermore, the testing program includes monotonic and cyclic tests of the walls, as well as screw connection assembly tests in double shear. In addition, finite element models of the shear walls were developed via the software ABAQUS and calibrated with the experimental results. To overcome convergence issues, the explicit solver was employed, and a linear kinematic hardening user-defined material model (VUMAT) was used. Finally, to assess the behavior and structural efficiency of the wall, numerous parametric studies were carried out. Several construction details were assessed, including height-to-width aspect ratio, spacing of screws, thickness of the framing members and end conditions of the wall assembly. The results indicate that the shear wall configuration discussed in this paper can reach capacities that are two times more than conventional CFS shear walls that are stipulated in current AISI S400 standard.

1. Introduction

Applications of cold-formed steel (CFS) framing members as main structural systems have gained wide acceptance in recent decades. Owing to its durability, sustainability, and high strength-to-weight ratio, CFS shear walls have emerged as an innovative and cost-effective seismic force resisting system (SFRS). A conventional CFS steel sheathed shear wall is comprised of CFS tracks and studs (typically C-sections), hold-downs to resist the overturning and uplift forces, and a steel sheathing fastened to one side or both sides of the frame using self-drilling screws [1]. The system dissipates energy through a combination of screw-bearing deformations and shear buckling of the sheathing. While conventional CFS shear walls have been researched extensively, current design standards restrict their applications to low-rise and mid-rise construction. For instance, the National Building Code of Canada [2] limits the height of CFS structures to 20 meters. Moreover, the AISI S400-20 [3] standard recommends design values for steel sheathed shear walls, unfortunately, the design values are limited by member thicknesses and screw spacing. Hence, to push the state-of-the-art, there is a pressing need for proposing novel CFS shear wall configurations that can attain higher strengths and ductility.

The focal point of this paper is to establish and validate benchmark shell finite element models of an innovative, higher-capacity CFS shear wall. In order to achieve that, a preliminary testing program was conducted that consisted of monotonic and cyclic tests on two full-scale CFS steel sheathed shear walls. Moreover, to gain insight on the walls' behaviour, shell finite element models were developed using the finite element software package ABAQUS [4]. Two benchmark models were established and calibrated against the experimental results. Finally, several parametric studies were undertaken to improve the performance of the shear wall.

One of the early attempts to quantify the lateral resistance of conventional CFS shear walls is the work performed by Serrette et al. [5]. The study involved a series of monotonic and cyclic tests on steel-sheathed and X-braced CFS shear walls. The design values were incorporated in older versions of the AISI S213 standard (2004). To address the lack of design provisions for CFS steel-sheathed shear walls in

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Canada, an extensive testing program was undertaken by Rogers et al., Balh & Rogers, and Ong-tone & Rogers [6-8]. 54 specimens with varying sheet and frame thicknesses, construction details and aspect ratios were subjected to monotonic and reverse cyclic loadings. The data was then analyzed using the equivalent energy elastic plastic method (EERP) in order to come up with resistance and seismic modification factors that are compatible with CFS design standards. The results of this research were also included in previous versions of AISI S400. More recently, another experimental program was completed by Santos & Rogers, Briere & Rogers, and Wu [9-11]. The main aim of this program was to achieve higher capacities and drift levels. Two original shear wall configurations were constructed and investigated, a double sheathed wall with built up box-studs, and a center-sheathed shear wall where the sheathing is confined between built-up framing members. The authors also proposed a preliminary design method, called the modified effective strip method (MESM), to calculate the nominal shear resistance of the walls. The main findings of this work indicate that the center-sheathed wall reached shear capacities that are four times higher than the design values listed in the code.

Regarding numerical analyses, Schafer et al. [12] presented a general overview of computational modelling of cold-formed steel members. Their work was concerned with assessing different modeling parameters and their effects on the collapse behavior of CFS isolated structures. In particular, the study incorporated considerations for geometric imperfections, mesh density, element selection, constitutive material model, and finally, the solution scheme. The main findings indicate that quadratic elements perform better than linear elements. Moreover, to reproduce all buckling modes (global, local, and distortional), a medium or a fine mesh is required. Zhang and Schafer [13] developed a model using ABAQUS to simulate the double-sheathed wall that was tested by Briere & Rogers [10]. The sheathing and framing members were modeled using quadratic shell elements with reduced integration (S8R) and the assembly was meshed with a fine mesh equivalent to a global seed of 0.5 inch. The sheathing-to-framing connections were simulated using horizontal and vertical linear elastic springs with fixed coupling in the out of plane direction. The authors reported that model’s response is qualitatively similar to the test, however a more robust screw model is required to match the experiments’ results. Since the response of CFS shear walls is greatly influenced by the sheathing-to-framing connections, Ding [14] conducted a comprehensive study on modeling monotonic and cyclic response of screw fastened connections. One of ABAQUS’s limitations is the lack of a connector element that can model the hysteretic behavior of self-tapping screws. Accordingly, Ding [14] programmed and tested a user element subroutine (UEL) for a nonlinear hysteretic model capable of simulating pinching and strength and stiffness degradation for CFS fastened connections. The OpenSees Pinching4 material model was converted to a user subroutine and implemented in ABAQUS shear wall models. All simulations were consistent with the shear-deformation response of recent monotonic and cyclic experiments confirming the validity of the UEL and displaying its potential in performing future cyclic simulations of CFS screw-fastened shear walls.

2. Experimental Program

The shear wall specimens investigated herein are constructed by concentrically placing a thin steel sheet between built-up studs at each end and built-up horizontal tracks at the top and bottom of the wall. Two hat sections are attached face-to-face to form box shaped studs, while the horizontal elements consist of two L-shaped angles that are fastened back-to-back. Hold-downs are connected to the exterior of the studs using high-strength bolts; and are placed at the top and bottom of the assembly. The specimen is schematically shown in figure 1. This specific configuration boasts several advantages when compared to conventional CFS shear walls. Sandwiching the sheet between box-like members reduces out-of-plane and torsional forces on the sheet and studs, thus reducing the severity of several failure modes that were encountered in previous research such as sheathing pull-through and torsional buckling of the end studs [8,9]. Furthermore, box members have higher axial and flexural capacities as well as higher torsional rigidity when compared to standard open CFS channel sections. Hence, it is predicted that the system will exhibit higher ductility and resist larger forces due to the increased screw bearing deformations.

![Figure 1: Structural details of shear wall specimen](image-url)
The testing program was conducted in Chongqing University under the supervision of the second author of this paper. The shear wall assemblies were mounted on a rigid frame that is equipped with an actuator capable of delivering a ±300mm horizontal stroke and a displacement-loading speed of 0.1~3 mm/s. The specimens are connected to the testing frame through the hold-downs and tracks. The hold-downs are attached to the top control beam and the bottom base beam through M22 high strength anchor bolts, while the tracks are connected to the beams using M12 anchor bolts. In addition, a rolling support is attached to the top control beam to restrict the wall’s out-of-plan movement. The test setup and specimen are shown in figure 2.

![Shear wall specimen and test frame setup.](image)

### Table 1: Testing matrix.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Sheet Thickness*</th>
<th>Wall Length*</th>
<th>Wall Height*</th>
<th>Frame Thickness*</th>
<th>Loading Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>GD-6</td>
<td>0.8</td>
<td>1210</td>
<td>2400</td>
<td>2.5</td>
<td>Mono.</td>
</tr>
<tr>
<td>GD-19</td>
<td>0.8</td>
<td>1210</td>
<td>2400</td>
<td>2.5</td>
<td>Cyclic</td>
</tr>
</tbody>
</table>

*All dimensions are in mm

The specimens were subjected to two displacement-controlled loadings: a monotonic (static) test and a cyclic test. As soon as lateral displacement was applied to the top of the wall, the tension field started developing due to the elastic shear buckling of the sheathing (even at displacements as low as 6 mm). At larger displacements, the tension bands became more defined, and the shear buckling had become inelastic at this point. For the cyclic tests, the tension field developed across both diagonals due to the load reversals. In both tests though, the controlling failure was the flexural buckling of the built-up studs. In the context of seismic design, chord stud failure in a CFS shear wall is undesirable since it is detrimental to structural integrity of the system. Regrettably, the rigid connection between the studs and the hold-downs that possess significantly higher stiffness led to the development of undesirable bending moments. Furthermore, the placement of the hold-downs to the exterior of the chord studs caused eccentricities that resulted in even higher bending moments on the studs. This can be clearly observed in figure 3 where the buckling and fracture of the chord studs occurred just above the hold down. Conversely, no screw failures were reported for the monotonic test, although two type of connections failure was observed in the cyclic tests: bearing failure of the sheathing and framing, and shear failure of the hold-down bolts. At higher displacement levels and after the studs have buckled, the remaining energy was being dissipated via the bearing damage of the sheathing and chord studs around the fasteners. The sheathing was prevented from pulling through since it was restrained between both flanges of the studs. The testing program also included coupon tests and connection assembly tests to determine the material properties of the steel and the backbone strength curve of the screws; however, they are outside the scope of this paper. The failure modes are shown in the figure 2.

### 2.1 Shear Wall Test Results

The test results of the monotonic and cyclic experiments are shown in table 2. The results are reported in terms of ultimate resistance (peak load), total energy dissipated, and drifts at ultimate resistance. Additionally, results from specimen tested by Santos & Rogers, Briere & Rogers, and Wu [9-11] are incorporated. For a fair assessment, specimens W34-CR3 and W15F-CR3 are included in the comparison since both specimens had similar configuration to the walls tested herein. It’s worth mentioning also that even though the specimens retrieved from [9-11] used thinner sheathing (0.42 mm), the specimens were reinforced by adding two extra chord studs to the frame of the wall in order to avoid premature buckling of the chord studs. As shown in table 2, the shear wall specimen had a lower capacity than the walls tested by [9-11] by 36%, although examining the total energy dissipated indicates that the walls tested in this paper have higher ductility as they dissipated more energy (43% increase). Furthermore, the experimental results indicate that the specimen can reach capacities that are two times more than conventional shear walls that are stipulated in the current standard [3].

### 3. Finite element analysis

The monotonic and cyclic tests of the concentric-sheathed CFS shear wall offer valuable insight on the hysteretic and
collapse behavior of the system. Due to the expensive nature of full-scale tests however, the range of building parameters that can be investigated is limited. Fortunately, finite element analysis offers flexibility in conducting various parametric studies that cover a wide range of building constraints and loading conditions. In this work, the commercial software package ABAQUS [4] was used to model and analyze the shear walls. This section outlines the basic modelling approach which includes the solution method, element type, material model, contact interactions, and other modelling assumptions.

3.1 Obtaining a quasi-static solution with ABAQUS/Explicit

Since the tests on the shear wall specimens were conducted under quasi-static conditions, initially the models were analyzed with the static general and the quasi-static dynamic implicit solvers. Both solvers however encountered several convergence issues and the analyses were terminated before the full displacement-load can be applied. At high drift levels, the solver reported negative eigenvalues and a severe cut back in the time increment. Negative eigenvalues are often associated with a loss of stiffness which physically might translate to the initiation of buckling. Indeed, the development of tension field due to elastic shear buckling of the sheet and the flexural buckling of the chord studs creates local and global instabilities which make it difficult to trace the solution past the elastic region using the implicit solver. The issue would be further exacerbated once cyclic loading is involved as the load reversal would create even more complex instabilities while the tension field readjusts itself during load/unload cycles [15]. Several approaches were employed to alleviate the convergence issues, including simplifying the model by using rigid parts, relaxing the convergence criteria, applying artificial damping, etc. For instance, ABAQUS offers the option of applying an adaptive stabilization scheme by adding artificial damping to the model to overcome convergence problems. Unfortunately, the artificial damping factor calculated by the software to remove instabilities was extremely high that it changed the physics of the problem. Consequently, and because the solver cutback often resulted in increment sizes less than (1.0e-6) seconds the explicit solver was applied, since a time increment of this size is suitable for an explicit time integration procedure. While the explicit dynamic procedure is commonly used to analyze brief transient dynamic events, it can be used to solve quasi-static problems as long as the kinetic energy is controlled. Perhaps the main advantage of using the explicit solver is the integration scheme. Whereas implicit analysis uses a stiffness-based, iterative procedure (such as Newton-Raphson) to solve a set of coupled nonlinear equations; the explicit solver takes advantage of a lumped mass matrix and employs a central difference operator to advance the kinematic state. As a result, no iterations are required and inverting the stiffness matrix becomes a trivial operation. Hence the explicit solver handles complex contact interactions and complex post-buckling problem efficiently. The major drawback of the explicit procedure is the time increment. Specifically, the time increment needs to be less than a stable time increment or the solution becomes unbounded. If no damping is present, the stability limit can be defined as:

$$\Delta t \leq \frac{2}{\omega_{max}}$$  \hspace{1cm} (1)
where \( \omega_{\text{max}} \) is the highest element frequency of the assembly. The estimate for the element highest frequency on the other hand is based on the dilatational mode of the mesh and is given by the following relations:

\[
\Delta t_{\text{stable}} \leq \frac{L_e}{C_d} \\
C_d = \frac{\sqrt{E/\rho}}{
\begin{equation}
\tag{2}
\end{equation}
\begin{equation}
\tag{3}
\end{equation}
\]

where \( L_e \) is the smallest characteristic length of the element, \( E \) is the modulus of elasticity, \( \rho \) is the density of the material, and \( C_d \) is the dilatational wave speed of the material. From equations (2) & (3), it can be concluded that the size of the mesh and material properties are the main factors that control the size of the stable time increment. Fortunately, an economic solution can be obtained by using one of the following approaches: mass scaling or increased loading rate. Mass scaling involves increasing the stable time increment by artificially increasing the density of the material; while increasing the loading rate artificially reduces the time scale of the process and thus fewer increment are required to complete the simulation. Regardless of the method used, it is critical to verify the validity of the static solution and that can be achieved by satisfying the energy balance equation (as discussed in the following subsections) and monitoring the kinetic energy.

### 3.2 Benchmark model details

Figure 4 shows a discretized assembly of the shear wall specimen in ABAQUS. The CFS framing members, hold-downs and steel sheathing are modeled using the S4R shell element. S4R is a 4-noded, general-purpose conventional shell element with six degrees of freedom, linear shape functions and a reduced integration scheme. Reduced integration elements significantly reduce the computational time of the analysis however they are prone to hourglassing, which is a zero strain, non-physical deformation mode. To eliminate this mode and to be consistent with the findings of Schafer et al. [12], the framing members and hold-downs were discretized with a fine mesh consisting of a global seed size equivalent to 10 mm, while the sheathing was meshed with a global seed size corresponding to 20 mm. This meshing technique ensures that two elements were meshed on the tracks’ lips and at least four elements were meshed on the studs’ outer flanges. One of ABAQUS/Explicit’s shortcomings is that it contains a rather simple material library. For instance, the software includes three hardening models for metals: isotropic hardening, perfect plasticity, and Johnson-Cook hardening model. The isotropic hardening model is suitable for modeling the monotonic test. However, when cyclic analysis is involved, and the specimen is subjected to stress/strain reversals, modeling the material’s strength and stiffness degradation becomes significant (Bauschinger effect). Fortunately, ABAQUS allows users to implement any mechanical constitutive material model through a user subroutine. Accordingly, a user-defined material (VUMAT) was retrieved from [15] to implement linear kinematic hardening. The kinematic hardening flow rule states that under cyclic loads, the yield surface undergoes rigid body translation (as opposed to expansion/contraction in isotropic hardening). The model requires basic material properties as input such as modulus of elasticity and yield strength, as well as a hardening parameter to define the back stress. The hardening parameter was obtained by first transforming the average true stress/strain curve of the steel to a bilinear curve and then taking the slope of the line connecting the yield stress to the ultimate stress. The screw connections were modeled using the cartesian connector element. This element represents each connection by three non-linear springs, two shear spring and one withdrawal spring. The backbone strength curve of the screws was obtained from the subsequent connection assembly test in the experimental program.

In order to imitate the rigid boundary condition that was provided by the base of the test frame, the nodes at the location of the anchor bolts and bottom hold-downs were pinned. On the other hand, to simulate the control beam, nodes at the locations of the anchor bolts at the top tracks and top hold-downs were tied to a reference point defined at the centroid of the top track using a kinematic coupling constraint. Since a roller support was provided in the physical test, the nodes at the top track were restrained from moving in the out-of-plane directions. A displacement was then applied to the reference point in the horizontal direction. To reduce noise and conduct quasi-static analysis efficiently, loads should be employed in a manner such that
the change of acceleration is minimal from one increment to another. This can be achieved in ABAQUS by using the SMOOTH STEP function. This particular function connects amplitude data pairs with curves whose first and second derivatives are zero.

When a structural system is subjected to a cyclic load, if the loading's frequency is less than nearly one quarter the system's lowest natural frequency [16], then the problem can be classified as a quasi-static one. According to the ABAQUS user manual, a lower bound time period for conducting quasi-static analysis with the explicit solver can be estimated by setting the loading duration to 10 times the period of the slowest mode. Subsequently, a frequency extraction step was defined where the lowest frequency of the model was obtained (figure 5). Then, to be conservative, the loading duration was set to 50 times the period of the first mode. As shown in the following figure, the loading duration of the monotonic analysis was set to $50 \times 1/4.75 = 11$ seconds. For the cyclic load, the loading duration between successive peaks was set to 5 seconds. Finally, mass scaling was applied to expedite the analysis. It was concluded that a constant mass scaling factor of 625 applied to the whole model would result in an economic solution with negligible inertial effects.

![Figure 5: Frequency extraction.](image)

4. Validation of benchmark models

The results of the cyclic and monotonic finite element analysis of the shear wall specimens are discussed in this section. The energy output curve for specimens GD6 and GD19 are shown in the following figures. The validity of the quasi-static solution is demonstrated by examining the energy plots. It can be observed that the external work and internal energy for both models are identical (grey curve and dotted curve) which implies that the total energy in the model is almost zero. In addition, the ratio of kinetic and artificial energies to the internal energy was about 5% and 7%, respectively. This confirms that the simulation was completed under quasi-static loading conditions and also indicates that the energies associated with viscous dissipation and hourglass control are negligible.

![Figure 6: Energy Output for specimen GD-6 (monotonic).](image)

![Figure 7: Energy Output for specimen GD-19 (cyclic).](image)

The force-displacement curves for the numerical and experimental results are shown in figures 8 & 9. For the monotonic test, the model estimates the strength of the system accurately as there is a minor difference between the experiment and numerical results (about 2%). Additionally, the model predicts the drift at peak loads reasonably well. The post-peak strength degradation of the system however is not reproduced accurately by the model.
This can be attributed to the material model as the VUMAT simplifies the stress-strain curve to a bilinear curve and the full range of effective plastic stress versus effective plastic strain is not considered.

Figure 8: Experimental vs. FEM results for the monotonic test.

Figure 9: Experimental vs. FEM results for the cyclic test.

The same observations can be made for the cyclic test. Generally, there is good agreement in terms of peak load and drifts at maximum base shear between the model and experiments, as displayed in table 2. Also, figure 10 shows the deformed shape of the model at various stages of the simulation and its evident that several failure modes are successfully captured.

Figure 10: Comparison of various failure modes between the numerical model and the experiment.
Table 3: Comparison of structural properties between the experiment and ABAQUS model

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Max Load (Exp.)</th>
<th>Max Load (FEM)</th>
<th>% Error</th>
<th>Drift at max load (Exp.)</th>
<th>Drift at max load (FEM)</th>
<th>% Error</th>
<th>Energy (Exp.)</th>
<th>Energy (FEM)</th>
<th>% Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>GD-6</td>
<td>102.5 kN</td>
<td>99.8 kN</td>
<td>2.6%</td>
<td>84 mm.</td>
<td>77 mm.</td>
<td>8.3%</td>
<td>18678</td>
<td>20080</td>
<td>7.5%</td>
</tr>
<tr>
<td>GD-19</td>
<td>105.3 kN</td>
<td>99 kN</td>
<td>6%</td>
<td>71.9 mm.</td>
<td>66.5 mm</td>
<td>7.5%</td>
<td>68241*</td>
<td>94330</td>
<td>38.2%</td>
</tr>
</tbody>
</table>

* Energy calculated for peak cycles only.

The efficiency of the model however is emphasized when the hysteretic properties of the system are evaluated. Referring to figure 9, the model simulates the loading and unloading stiffness of the system with reasonable accuracy. Moreover, comparisons of isolated hysteretic loops are displayed in figure 11 and the Bauschinger effect is clearly replicated by the model. On the other hand, both models overestimate the initial stiffness of the system. It was reported that the screws were not snug tight at the beginning of the experiment and required the application of the displacement-controlled load in order for the fastened members to be firmly connected to each other. Hence the ‘slip’ of the screws is reflected in the pre-yield portion of the numerical graphs where the model’s response is stiffer than the experiment. The main sources of error in this work can be credited to the energy dissipation mechanism. As previously stated, a CFS shear wall dissipates energy through yielding of the sheathing and screw bearing deformations. Therefore, from a numerical point of view, the sources of stiffness and strength degradation in the model are attributed to the constitutive material model and the choice of connector element. The linear kinematic hardening model is somewhat successful in modelling the hysteresis loops; nonetheless, to model the full severity of the Bauschinger effect, two adjustments are required: (a) define a more robust kinematic hardening model, and (b) model the screws using a connector element that is capable of simulating pinching and strength degradation parameters is essential. The current connector elements in the ABAQUS library either define a screw backbone curve using pairs of force-deformation (springs) or establish connections by eliminating degrees of freedom (multi-point constraints).

5. Parametric studies

The benchmark models have been validated against the experimental results, and the model shows good agreement in terms of peak load, failure modes and load-displacement behavior. In this section, several construction details and parameters are assessed in order to improve the performance of the shear wall. The parameters investigated herein include: 1) framing thickness, 2) screw spacing, 3) aspect ratio, and 4) placing hold-downs on one/two sides of the studs.

5.1 Framing thickness

To study the influence of the thickness of the framing members, four models were setup with various stud and track thicknesses. Based on local manufacturer’s specifications, the thicknesses that were assessed were 3.05 mm, 3.43 mm, 3.81 mm, and 4.2 mm. The load-displacement curves of the different models as well as the benchmark model and experiment are shown in figure 12. As predicted, the strength and stiffness of the wall increases as the framing thickness increases. Interestingly, models of frames thicker than 3.05 mm show no obvious material softening/degradation point at high drifts (as opposed to the benchmark model and the 3.05 mm model). Moreover, further examination of the deformed shapes indicates that as the framing members get thicker, the end stud buckling...
failure mode is eliminated and the dominant failure mode is the inelastic shear buckling of the sheet.

\[ \text{Displacement (mm)} \quad \text{Force (kN)} \]

| Experiment | 3.05 mm | 3.43 mm | 3.81 mm | 4.2 mm |

Figure 12: Comparison of shear wall models with varying framing thickness.

5.2 Wall Height-to-Width Aspect ratio

The influence of the wall height-to-width aspect ratio is examined next. The original specimen had an aspect ratio of 1.98, to stay consistent with building specifications, the height of the wall was kept constant, and the width of the wall was varied. Referring to figure 1, the sheet is fastened to both flanges of the built-up studs. Hence for the first configuration, the width of the wall was increased by moving the studs to the outer parameter of the wall and attaching the sheet to the inner flanges only. For the second configuration, the distance between the studs remained the same however the portion of the sheet that was connected to the out flange was removed. By doing that, the width of the sheathing was reduced from 1210 mm to 890 resulting in a panel aspect ratio of approximately 2.7. The motivation behind the choice of configurations is to either reach higher capacities with the same amount of material or simply reduce the amount of material to optimize the economy of the shear wall specimen. The results of the cyclic analysis for both configurations are plotted against the experimental results in the following figure. According to figure 13, increasing the out-to-out dimension of the wall results in a 9% increase in peak load, while reducing the width of the sheathing results in an 8% decrease in peak load. For both configurations however, there is a significant decrease in the loading and unloading stiffness of the model. Attaching the sheathing to both flanges creates a semi-rigid vertical boundary for the panel. Conversely, attaching the sheathing to one flange, regardless of width, results in a more flexible boundary and similarly results in a decreased unloading/loading stiffness.

\[ \text{Displacement (mm)} \quad \text{Force (kN)} \]

| Narrow | Exp. | Wide |

Figure 13: Cyclic analysis for walls with varying aspect ratios.

5.3 Screw spacing

Three different patterns of screw spacings were investigated: 25mm, 75 mm, and 100 mm. As displayed in figure 14, it can be concluded that under this specific configuration, the influence of the spacing of the screws is minor. Further examination of the force-displacement plots shows that the plots of model with 75mm and 100mm are approximately the same. On the other hand, there is an insignificant increase in the peak load when the spacing of the screws is limited to 25 mm (1.7%). Currently, other configurations and details are being investigated in order to maximize the efficiency of the screw's contribution to the overall resistance of the wall.

\[ \text{Displacement (mm)} \quad \text{Force (kN)} \]

| Experiment | 25 mm | 100 mm | 75 mm |

Figure 14: Comparison between different screw spacings.

5.4 Screw spacing using different configurations

In order to assess the degree of screw contribution to the resistance of the shear wall, a parametric study involving the narrow wall discussed in section 5.2 was performed. In this
section, the width of the sheathing is reduced to 890 mm and the sheathing is attached to one flange of the built-up chord studs. Then, additional screw spacings were examined: 25 mm, 50 mm, 75 mm, 100 mm and 150 mm. The main objective was to keep the aspect ratio of the wall constant and examine if attaching the sheathing to one flange would have an effect on the screw spacing contribution. As displayed in figure 15, attaching the sheathing to one flange of the built-up studs yields a system that behaves similarly to conventional shear walls. It can be observed that when using such a configuration, the shear wall’s elastic stiffness increases as the screw spacing decreases. Conversely, it can be concluded that as the screw spacing increases, the drift at maximum load increases. As the screw spacing is decreased, the forces on the end studs increase which lead to premature chord buckling. This can be reflected by examining the graph of the model with 100 mm screw spacing where the peak load occurs at a drift equal to 125 mm whereas the peak load for models with screw spacing less than 100 mm occurs at drifts equal to 105 mm.

![Figure 15: Comparison between different screw spacing for the narrow wall.](image)

### 5.5 Hold-down placement

A model was setup with the hold-downs attached to both sides of the studs. The main aim was to reduce the bending induced by the eccentricity associated with the placement of hold-downs on one side of the chord studs. Minor design adjustments were required to fit the hold-downs on both sides of the studs. As displayed in figures 16 and 17, the deflected shape of the studs slightly resembles the deformed shape of a column with fixed boundaries on both ends. Also, placing the hold-downs on both sides resulted in even higher concentration of stresses at the face of the hold-downs and led to the development of a more pronounced tension field, indicating an increase in the rigidity of the frame. Fortunately, there is a substantial increase in the peak load (20%), energy dissipated and stiffness of the system. Hence while the issue of the chord stud buckling is not entirely resolved, the high capacities and drifts dissipated by the system shows potential.

![Figure 16: Deformed shape of the FEM models.](image)

![Figure 17: Comparison of force-deformation curves.](image)

### 6. Conclusions

In this paper, benchmark finite element models of a novel, high-capacity cold-formed steel shear wall are developed and calibrated against experimental results. The main conclusions can be summarized as follows:

- The results of the shear wall monotonic and cyclic test show potential in expanding the application of shear walls to mid-rise buildings. When compared to walls using similar configurations from the literature, the shear wall tested herein displayed higher ductility and attained high forces without the requirement of reinforcing the frame.
- The experimental results also indicate that the specimen can reach capacities that are two times more than conventional shear walls that are stipulated in the current standard.
- The controlling failure mode is the flexural buckling of the end studs. Such a failure mode is not desirable for an SFRS, and hence numerical parametric studies have been undertaken to improve the performance of the wall.
- The monotonic shell finite element models show good agreement with the experimental results in terms of deformed shape, ultimate load, drifts, and energy dissipated.
- The cyclic model overestimates the energy dissipated by the system, hence, a more robust material model that incorporates kinematic hardening is recommended.
- Regarding the parametric studies, it is concluded that the framing thickness has a significant influence on the performance of the wall.
- At thicknesses higher than 3.05 the framing members behave rigidly and the stud buckling failure mode is eliminated while the controlling mode of failure is the inelastic shear buckling of the sheathing.
- Increasing the out-to-out dimension of the wall increases the ultimate capacity by 9%. While reducing the aspect ratio of the panel reduces the capacity slightly however less sheathing material is required.
- The influence of the spacing of the screws is minor under the current configuration. However, the influence is more evident when the sheathing is attached to one flange only.
- Attaching the hold-downs to both sides of the wall substantially increases the capacity of the wall.

References