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Wei-Wen Yu International Specialty Conference on Cold-Formed Steel Structures
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CHARACTERIZATION OF COLD-FORMED STEEL FRAMED DIAPHRAGM RESPONSE UNDER IN-PLANE LOADING AND INFLUENCE OF NON-STRUCTURAL GYPSUM PANELS

Patrick Latreille[1], Violetta Nikolaidou[2], Colin A. Rogers[3], Dimitrios G. Lignos[4]

Abstract

The in-plane response of CFS framed diaphragm structures subjected to seismic excitation is not well understood. At present, the North American AISI S400 Standard does not include a seismic design procedure for CFS framed diaphragms for use in Canada, and offers limited information for their use in the US. In addition, the effect of non-structural components on the lateral strength and stiffness of the diaphragm component has yet to be explored. In an effort to provide insight into the complex nature of the diaphragm structure and the influence of non- structural components an experimental program was initiated in the Jamieson Structures Laboratory at McGill University focusing on the characterization of the behaviour of CFS framed - wood sheathed diaphragms under in-plane loading. This paper presents the results for four diaphragm configurations with oriented strand board sheathing (OSB) tested under

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monotonic and reversed cyclic loading following the cantilever test method. The 3.7m x 6.1m diaphragm specimens were constructed with different structural configurations as well as non-structural gypsum panels below the steel framing. Design predictions for the shear strength and deflection of the diaphragm specimens were obtained using the information available in the AISI S400 Standard.

Introduction

Currently, the design of the lateral force resisting systems of cold-formed steel framed structures revolves largely around shear walls, for which extensive experimental and numerical work has been conducted, e.g. Liu et al. 2012, Shamim 2012, Peterman 2014, among others. While shear walls are well understood, there is little research that exists on the diaphragm's contribution to the overall seismic response of the CFS structure. At present, in Canada no design provisions exist for CFS framed diaphragms. In the US, there exist limited resources in the current seismic code provisions that are based largely on experimental work done on wood diaphragms and shear walls (AISI 2015). In addition, the effect of non-structural components such as gypsum panels on the overall lateral stiffness of the CFS diaphragm has yet to be investigated. Therefore, the need to address these design deficiencies is evident in order to assist professional engineers in the construction of safer and more economical CFS structures.

One of the first research projects focusing on the lateral response of CFS framed diaphragms was conducted by the National Association of Home Builders Research Center (NAHBRC 1999). Their experiment-based research provided shear strength and stiffness values for four diaphragm configurations. Lum's analytical work provided allowable design shear strength values for a limited number of CFS framed / plywood sheathed diaphragm configurations (LGSEA 1998). These values are available in Table F2.4-1 of the AISI S400 Standard (2015). A deflection equation, developed by Serrette and Chau (2003) is also available in the S400 standard for both shear walls and simply supported diaphragms.

In recent years, it was the work conducted by researchers at Johns Hopkins University that provided a better insight in the overall lateral response of CFS structures (Peterman 2014). The CFS - NEES project involved the investigation of the overall seismic response of a two storey CFS framed structure subjected to earthquake loading (Liu et al. 2012, Peterman 2014). After the completion of these tests the importance of obtaining more information concerning the isolated seismic performance of the diaphragm subsystem as well as including the effect of non-structural components was noted. Gypsum's contribution to the overall response was demonstrated in the numerical work of Shamim and Rogers (2013, 2015), which showed that adding a single 12.5mm gypsum layer to the steel sheathed shear walls of a CFS framed structure led to an increase of the overall seismic capacity and a favourable change in the response to ground motions. In addition, test results of CFS strap braced walls by Lu (2015) showed that installing two layers of gypsum on both sides of a wall to achieve a two hour fire resistance rating can nearly double the ultimate shear strength of this lateral load carrying system.

In 2015 a total of eight diaphragm tests were conducted at McGill University in order to characterize the behaviour of CFS framed / OSB sheathed roof and floor structures under in-plane monotonic and reversed cyclic loading. The tests were based on the configurations used in the CFS – NEES building and were conducted using a cantilever diaphragm test apparatus with 3.66m x 6.10m specimens (Nikolaidou et al. 2015). The experimental work presented herein focuses on building upon these tests. In this paper the testing of three new diaphragm configurations is described, for which structural changes were made, in addition to a fourth configuration, to which non-structural gypsum panels were attached. All of the diaphragm configurations were tested under in-plane monotonic loading, while the specimen with non-structural gypsum panels was also tested under reversed cyclic loading. This resulted in four diaphragm configurations with a total of five tests performed. In addition to describing the testing and test results, this paper contains a comparison between the measured test values and the calculated deflection as well as shear strength values following the AISI S400 Standard (2015).

Test program

The test setup constructed to accommodate the diaphragm specimens is presented in Figure 1. It is of the cantilever configuration and was designed to perform as a self-reacting braced frame with W-shape sections chosen for the main beams and double angle sections for the bracing (Nikolaidou et al. 2015). The frame dimensions were chosen to be 4.5m x 6.5m, taking into account the space limitations of the Jamieson Structures Laboratory, which restricted the test specimen size to 3.66m x 6.10m.

In the previous experimental work of CFS framed diaphragms realized at McGill University (Phase 1), the first two tests performed were that of the bare frame to measure the corresponding stiffness of the underlying CFS structure (Nikolaidou et al. 2015). Following this, the basic roof and floor diaphragm configurations of the CFS – NEES building were tested (Table 1). Two additional test specimens were then included, in which a single structural alteration was featured. For the roof diaphragm, full panel blocking was added and for the floor diaphragm a larger screw size was used. Both monotonic and reversed cyclic tests were performed for each specimen resulting in a total of 10 tests. A thorough description of these Phase 1 tests is provided in the report by Nikolaidou et al. (2015). The Phase 2 research summarized in this paper is an extension of the laboratory study completed by Nikolaidou et al.. Tables 2 and 3 contain an inclusive list of nomenclature for all Phase 1 and 2 diaphragm specimens tested to date. The configurations documented in this report correspond to specimens 11 through 16 (Table 3), which are illustrated in Figures 2 through 5.

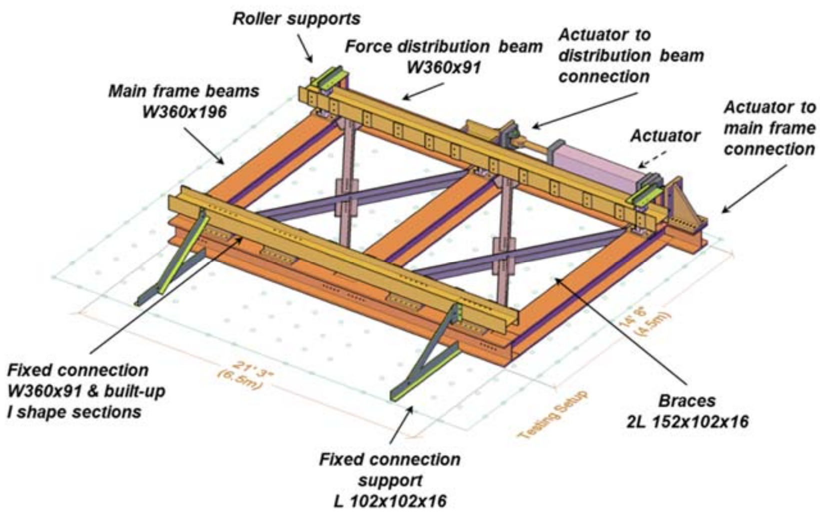

Figure 1 – CFS Diaphragm Test Setup

Table 1 – Floor and Roof Diaphragm Basic Configurations (Nikolaidou et al. 2015)

Element	Roof	Floor
Joists	1200S200-54	1200S250-97
Rim Joists	1200T200-68	1200T200-97
Web Stiffeners	L 38.1x38.1x1.37	L 38.1x38.1x1.37
Joist bracing	1200S162-54	1200S200-54
Joist bracing connectors	L 38.1x101.6x1.37	L 38.1x101.6x1.37
Joist bracing straps	38.1x1.37	38.1x1.37
Sheathing self-drilling screws (150mm/300mm spacing)	#8	#10/#12
OSB panels	2440x1220x11.11	2440x1220x18.25
#10 flat head self-drilling screws : all joist to rim joist flange connections		
#10 hex head self-drilling screws : all joist to rim joist web angle & joist bracing connections		

Table 2 – Phase 1 Diaphragm Specimen Nomenclature

Specimen	Description
1-RF-M	Roof Bare Steel Frame : Monotonic
2-FF-M	Floor Bare Steel Frame : Monotonic
3-RU-M	Roof Unblocked : Monotonic
4-RU-C	Roof Unblocked : Reversed Cyclic
5-F#10-M	Floor #10 Screws : Monotonic
6-F#10-C	Floor #10 Screws : Reversed Cyclic
7-RB-M	Roof Blocked : Monotonic
8-RB-C	Roof Blocked : Reversed Cyclic
9-F#12-M	Floor #12 Screws : Monotonic
10-F#12-C	Floor #12 Screws : Reversed Cyclic

Note: Tests completed by Nikolaidou et al. (2015)

Table 3 – Phase 2 Diaphragm Specimen Nomenclature

Specimen	Description
11-RALT-M	Roof Blocked Alternate Direction Joists : Monotonic
12-RSTRAP-M	Roof Strap Blocking : Monotonic
13-FB4-M	Floor #12 Screws Blocked (100mm/300mm) Spacing : Monotonic
14-RGYP-M	Roof with Gypsum Ceiling : Monotonic
15-RGYP-C	Roof with Gypsum Ceiling : Reversed Cyclic

(11-RALT-M) Roof Alternate Direction Joists - Monotonic Loading

Specimen 11-RALT-M was the same as the blocked roof specimen tested in 2015 (7-RB-M) with a 90 degree change in orientation of the joists (Figure 2). The main purpose of this configuration was to observe how the strength and stiffness would be affected if the applied load were parallel to the joists rather than perpendicular.

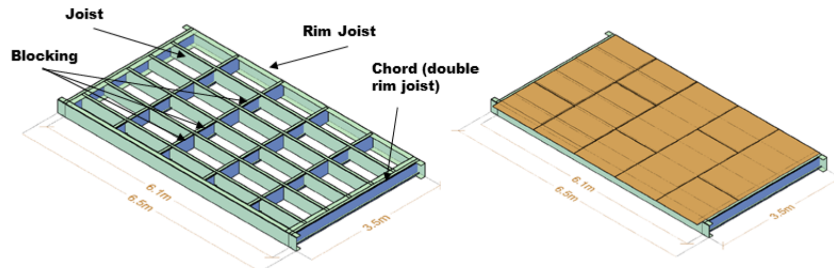

Figure 2 – Bare Frame and Frame with Sheathing : Test 11-RALT-M

(12-RSTRAP-M) Roof Strap Blocking - Monotonic Loading

Specimen 12-RSTRAP-M was also similar to 7-RB-M, with the exception that the full blocking at the OSB panel edges was replaced with strap blocking. Two lines of blocking were installed which were each composed of four fully blocked segments ("web stiffener" in figure) and a continuous steel strap on the top and bottom (Figure 3). The main purpose of this configuration was to determine if strap blocking, which is less costly and easier to install, would be as effective as full blocking in terms of providing adequate support to the OSB panel edges to attain similar diaphragm shear strength and stiffness.

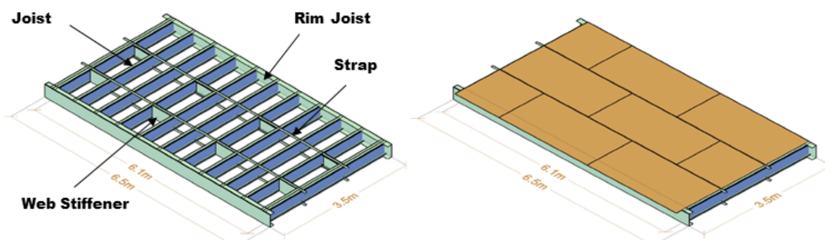

Figure 3 – Bare Frame and Frame with Sheathing : Test 12-RSTRAP-M

(13-FB4-M) Floor Blocked (100mm / 300mm) Spacing - Monotonic Loading

Test specimen 13-FB4-M (Figure 4) was designed to maximise the shear resistance of a diaphragm given a basic floor configuration of a 2.5 mm thick steel frame and 18.3 mm thick OSB sheathing. It was decided to use a fully blocked floor specimen with a screw (#12) spacing of 100mm along all panel edges. The primary objective of this configuration was to obtain an upper estimate for the design strength of these diaphragms.

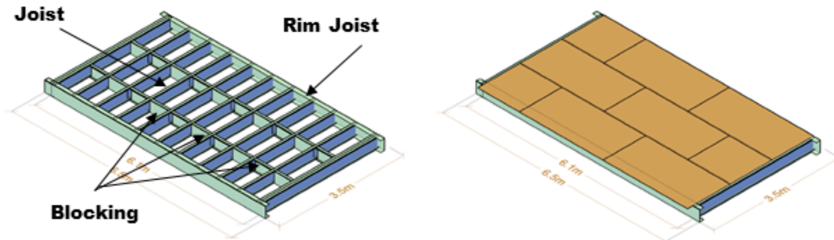

Figure 4 – Bare Frame and Frame with Sheathing : Test 13-FB4-M

(14-RGYP-M & 15-RGYP-C) Roof with Gypsum Ceiling - Monotonic & Cyclical Loading

Test specimens 14-RGYP-M and 15-RGYP-C comprised a basic roof configuration (3-RU-M) with one layer of type X, 16mm thick gypsum ceiling installed to the underside of the frame (Figure 5). This floor assembly is expected to attain a fire resistance rating of 45minutes to 1-hour (SFA 2013). The gypsum was directly attached to the underside of the CFS framing without the use of resilient channels; Lu (2015) showed that when resilient channels are used to attach gypsum panels to strap braced walls, the influence of the gypsum on the strength and stiffness is close to negligible. The fasteners used to attach the gypsum panels to the framing were #6 32mm long Type S drywall screws, spaced at 305mm o/c throughout (perimeter and field). Joint compound and joint tape were applied to the panel intermediate and screw locations in order to reinforce and conceal the joints and screw heads. The main purpose was to examine the contribution of the non-structural gypsum panels to the shear strength and stiffness of the diaphragm. For the reversed cyclic test the CUREE (Consortium of Universities for Research in Earthquake Engineering) reversed cyclic displacement controlled loading protocol was employed (Krawinkler et al. 2000).

It should be noted that the double joist shown at the ends of each diaphragm configuration in Figures 2 through 5 was placed in an effort to include the stiffening effect of a wall attached to the underside of the diaphragm. Also, to account for the ledger framing used in the CFS – NEES building, the sheathing had an extension of 152mm past the edge of the steel diaphragm frame to match the detail commonly used in construction; see Nikolaidou et al. (2015) for further construction details.

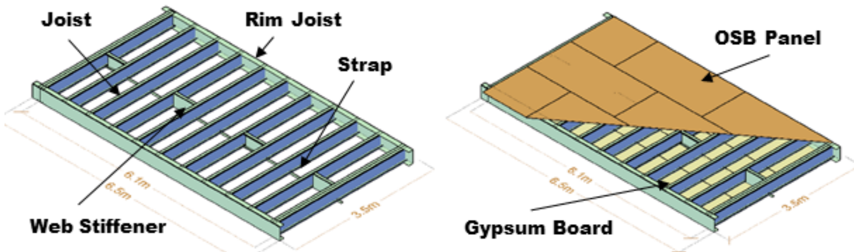

Figure 5 – Bare Frame and Frame with Sheathing & Gypsum : Tests 14-RGYP-M and 15-RGYP-C

Instrumentation

The instrumentation included four string potentiometers 254mm & 508mm stroke to capture the lateral displacement and overall shear deformation of each diaphragm, as well as eight linear variable differential transformers (LVDTs) ±15 mm stroke to measure the local in-plane displacement. The locations of the instruments are shown in Figure 6. In addition, the force on and displacement of the actuator were recorded. The measurement instruments were connected to Vishay Model 5100B scanners that were used to record data using the Vishay System 5000 StrainSmart software.

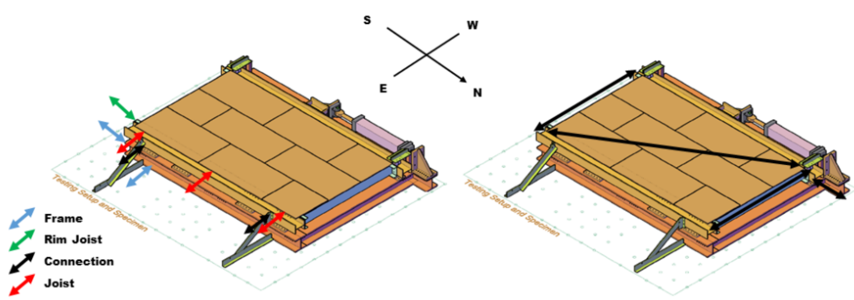

Figure 6 – Placement of LVDT sensors (left) and string potentiometers (right)

Test results

The results from the diaphragm tests for shear vs. deformation (rotation and displacement) response are presented in Figure 7. The alternate direction test 11-RALT-M as shown in Figure 7a and 8 had an unexpected failure in the chord to rim joist connection. The screws fastening the double joist, at the north end of the diaphragm, into the rim joist experienced shear failure. This is what caused the two sudden decreases in resistance prior to peak load (Figure 7a). The ultimate resistance was controlled by sheathing connection failure, i.e. tear out and pull through of the screw fasteners. In the post peak range, once the sheathing was nearly completely detached from the steel frame, a hinge in the CFS framing was developed as shown in Figure 8; which is consistent with how the 7-RB-M specimen failed. In the 7-RB-M test the in-plane uplift force generated at the north-east corner of the diaphragm was distributed amongst the joists spanning the east-west direction; as such, the chord to rim joist connection failure seen in test 11-RALT-M did not occur. However, in the 11-RALT-M test, because the joists were oriented in the north-south direction it was the segmented lines of blocking in the east-west direction and the north end chord that carried the in-plane uplift force. The blocking members and their end connections were significantly less stiff resulting in an increased load on the end chord. This increased load is believed to have caused the failure in the chord to rim joist connection. Nonetheless, the shear resistance of test 11-RALT-M is within close proximity to that achieved by test 7-RB-M; it is hypothesized that if this chord to rim joist connection had been designed to carry the full in-plane uplift force, without the aid of the interior blocking lines, the shear vs. deformation response would have been similar for these two diaphragm specimens. It is also relevant to note the importance of anticipating this in-plane uplift force and detailing the framing connections for the forces arriving from different loading directions on a building's diaphragm structures.

It is demonstrated in Figure 7b how comparable strap blocking (12-RSTRAP-M) is to full blocking (7RB-M) in terms of supporting the edges of the OSB panels and providing diaphragm shear resistance. The rigidity of both specimens were nearly identical, while their peak loads were within 10% of one another. The slightly increased peak load in the 12-RSTRAP-M case was most likely the result of minor changes in material properties of the OSB and CFS frame, which were sourced at different times. The ultimate shear resistance for both specimen with full blocking and strap blocking was related to the sheathing connection failures, as shown in Figure 9.

Specimen 13-FB4-M was designed to maximise the shear resistance of the floor diaphragm configuration. By fastening the edges of all OSB panels to frame blocking and by reducing the spacing of the sheathing screw edge fasteners to 100mm the maximum shear resistance was increased by over three times compared with the standard floor configuration (9-F#12-M) (Figure 7c). The reduced spacing of the sheathing fasteners increased the diaphragm shear rotation needed to cause failure by nearly double (Figure 7c). It was also the only test where the steel frame and sheathing failed together, rather than the sheathing connections first followed by the hinge action of the frame. The diagonal compression field that developed across the diaphragm caused noticeable damage to the underlying joists and blocking following a path between the south-east to north-west corners (Figure 9).

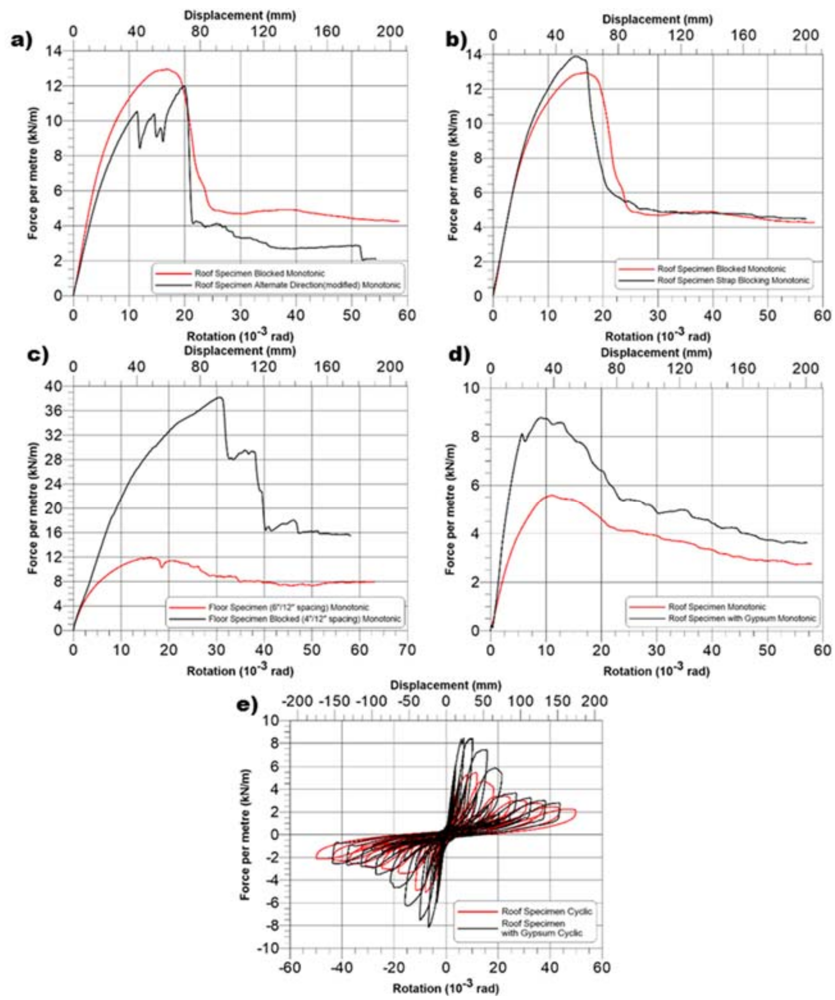

Figure 7 –Force vs. Deformation response for specimens: a) 11-RALT-M vs. 7-RB-M b) 12-RSTRAP-M vs. 7-RB-M c) 13-FB4-M vs. 9-F#12-M d) 14-RGYP-M vs. 3-RU-M e) 15-RGYP-C vs. 4-RU-C

Figure 8 – Test 11-RALT-M failure of chord to rim joist connection (left) and hinge created in the CFS frame after loss of the sheathing connections (right)

Figure 9 – Test 12-RSTRAP-M sheathing connection failures (left) and Test 13-FB4-M sheathing connection failures with compression field damage to steel frame (OSB panel removed for photograph) (right)

A 60% increase in shear strength and an approximate 105% increase in shear stiffness were experienced by tests 14-RGYP-M and 15-RGYP-C compared to the standard roof configuration specimens tested in Phase 1 due to the addition of the gypsum panels (Figures 7d & 7e). Overall, the diaphragms behaved in a similar fashion to that observed for the constructions without the gypsum panels, i.e. sheathing screw connection failures with lift-off of the OSB panels at the intermediate panel edge locations (Figure 10). The small drop in shear resistance (Figure 7d) just prior to the peak resistance for the monotonic test was due to some drywall screws in the gypsum to frame connections failing in shear. The reversed cyclic test was characterised, as expected, by a faster decline of the shear resistance after the peak load was reached compared to the monotonic response due to the accumulation of damage during the loading cycles.

Figure 10 – Test 14-RGYP-M OSB panel lift-off (left) and Test 15-RGYP-C sheathing connection failures (right)

Table 4 summarizes the measured results for the Phase 2 diaphragm tests.

Table 4 – General results from the Phase 2 diaphragm tests

Specimens	S_u (kN/m)	$\Delta_{net,o.4u}$ (mm)	$\Delta_{net,u}$ (mm)	$\theta_{net,u}$ (rad x 10^{-3})	Rigidity, K (kN/mm)
11-RALT-M	12.4	12.5	69.3	19.8	2.41
12-RSTRAP-M	14.2	11.7	52.4	15.0	2.96
13-FB4-M	38.7	22.8	108.7	31.0	4.14
14-RGYP-M	9.0	7.0	34.1	9.7	3.12
15-RGYP-C*	8.6	8.6	23.9	6.8	2.39

*Based on cycle during which the maximum resistance was reached

Design predictions

The AISI S400 Standard contains equation C-F2.4.3-1 obtain deflection design values for simply supported diaphragms (AISI 2015). However, in the current work Eq. E1.4.1.4-1 from AISI S400 (Eq. 1 in this paper) for blocked shear walls was deemed appropriate for comparison purposes with measured displacement due to the cantilever support conditions utilised in the diaphragm tests.

$$\delta = \frac{2vh^3}{3E_sA_cb} + \frac{\omega_1\omega_2 vh}{\rho G t_{sheathing}} + \omega_1^{\frac{5}{4}}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v \qquad (1)$$

A_c = Gross cross-sectional area of chord member (mm^2)
b = Width of the shear wall (mm)
E_s = Modulus of elasticity of steel 203000 MPa

G = Shear modulus of sheathing material (MPa)
H = Wall height (mm)
s = Maximum fastener spacing at panel edges (mm)
$t_{sheathing}$ = Nominal panel thickness (mm)
t_{stud} = Nominal framing thickness (mm)
v = Shear demand (V/b) (N/m)
V = Total in-plane load applied to the diaphragm (N)
β = 2.35 for plywood and 1.91 for OSB for SI units ($N/mm^{1.5}$)
δ = Calculated deflection (mm)
δ_v = Vertical deformation of anchorage / attachment details (mm)
ρ = 1.85 for plywood and 1.05 for OSB
ω_1 = s/152.4 ω_2 = $0.838/t_{stud}$
ω_3 = $\sqrt{((h/b)/2)}$ ω_4 = 1 for wood structural panels

Equation 1 uses empirical factors to account for inelastic behaviour, however, as found in Phase 1, these are proven to be inadequate for the diaphragm situation because they were formulated from shear wall tests largely composed of walls with a single wood panel (Nikolaidou et al. 2015). The only method where the results were comparable was to calculate and compare to the elastic deflection ($\delta_{ELASTIC}$). The elastic deflection is determined using the stiffness values taken from the 40% shear demand level, and extrapolating to the peak shear resistance. These results are summarised in Table 5.

Table 5 – Design deflection values (mm) Eq. 1 and comparison with $\delta_{ELASTIC}$

Diaphragm Specimens	11-RALT-M	12-RSTRAP-M	13-FB4-M	14-RGYP-M 15-RGYP-C
$\delta_{Calculated}$	45.2	32.1	62.6	17.9
$\delta_{ELASTIC}$	31.4	29.2	56.9	17.6
% Error	20.0	5.5	5.2	1.0

The AISI S400 Standard contains Table F2.4-1 to provide design shear strength values for a limited number of diaphragm configurations with CFS framing and plywood sheathing (AISI 2015). Table 6 lists the design shear strength values (**V_{design}**) to be considered for each diaphragm test configuration based on Table F2.4-1 and the measured shear resistance values (**V_{test}**). Note that Table F2.4-1 refers only to plywood sheathing and does not include the sheathing thickness used for the floor specimen (13-FB4-M). While these factors limit the ability to provide appropriate values based on the test configurations, at present, these are the only design values available for comparison with the test results.

Table 6 – Shear resistance design values using Table F2.4-1 AISI S400 (2015)

Diaphragm Specimens	11-RALT-M	12-RSTRAP-M	13-FB4-M	14-RGYP-M & 15-RGYP-C
V_{design} (kN/m)	11.1	11.1	25.8	7.4
V_{test} (kN/m)	12.4	14.2	38.7	9.0

Conclusions

The focus of this paper was to characterize the in-plane behaviour of four CFS framed/OSB sheathed diaphragm configurations under monotonic and reversed cyclic loading. The tests described herein are complementary to previous experimental work conducted at McGill University in 2015. This second phase of testing examined the effectiveness of the strap method as blocking, the effect of the joist orientation on the overall diaphragm response and aimed to obtain an upper threshold for shear strength and stiffness by testing a fully blocked floor configuration with 100mm screw spacing. In addition, the effect of gypsum as a non-structural component on the diaphragm response was also investigated. The direction of loading was shown to have little effect on the shear strength and stiffness of the diaphragm, assuming that the in-plane uplift forces are properly accounted for in design of the CFS framing and connections. Strap blocking of the OSB panel perimeters was shown to be just as effective as full blocking. Gypsum was shown to have a significant impact on shear strength and stiffness. In addition, using the shear wall deflection equation (Eq. 1 in this paper) of the AISI S400 Standard led to a meaningful comparison between the calculated and observed data only by assuming elastic response of the diaphragm. Lastly, the limited information available in the AISI S400 Standard did not allow for reliable design shear strength values to be obtained.

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